Current Development of Seismic Design Code to Consider the Near-fault Effect in Taiwan

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ABSTRACT

The object of this paper is to develop the design methods for structures against the near-fault ground motions. The first method is to consider the near-fault seismic demand at the design level, and hence, the near-fault design response spectrum is developed to intensify the resistance capacity of structures directly. The other alternative method is called the two-level design method. No near-fault effect is considered at the primary force-based design level, but the additional capacity checking level is requested to limit the ultimate capacity of the designed structure to exceed the maximum considered seismic demand caused by the near-fault effect.

INTRODUCTION

In recent years, people have learned that near-fault ground motions have many different characteristics from the far-field ones, and the near-fault ground motions will cause much more damage. In fact, the associated high PGA and the pulse-like velocity waveform of the near-fault ground motion will destroy structures with short and long structural periods, respectively [1~3]. Prior to the 1994 Northridge earthquake, the near source effects were particularly addressed by SEAOC for the UBC97 [4]. In UBC97, the near-source factors are incorporated in Seismic Zone 4, which is intended to recognize the amplified ground motions occurring close to the fault. Two near-source factors defined for the short period (acceleration control) and long period (velocity control) domains are needed because the effect is substantially greater at longer periods. After the 1999 Chi-Chi (Taiwan) earthquake, the normalized near-fault design response spectrum for sites near the Chelungpu Fault was developed based on the current Taiwan seismic design code and the near-fault attenuation functions for the spectral acceleration demands, which was regressed from the near-fault ground motions observed during the Chi-Chi earthquake [5].

The current design requirement for structures in Taiwan is based on the seismic hazard defined at a uniform 10 percent probability of exceedance in 50 years (return period of 475 years). Therefore, in order to consider the near-fault effect at the same seismic hazard level, the near-fault attenuation law corresponding to the magnitude of an event with expected recurrence rate of 475 years should be considered. However, it is much more difficult to estimate the magnitude of event with an expected recurrence rate of 475 years than to estimate the maximum potential magnitude. Then, to consider the near-fault effect, the so-called maximum considered earthquake (MCE) with 2 percent probability of exceedance in 50 years (return period of 2,500 years) should be taken into account [6].

For an interesting near-fault zone, both the probabilistic analysis based on the seismic hazard analysis at a return period of 2,500 years and the deterministic analysis based on the attenuation law corresponding to the maximum potential magnitude of the fault are implemented to define the seismic demands at the MCE level. The seismic demand should be dominated by the fault effect in the region with smaller distance from the fault where the seismic demand determined by the attenuation law is larger than that determined by the probabilistic analysis. Otherwise, the fault effect can be ignored and the seismic demand should be dominated by other potential sources. Therefore, as shown in Fig. 1, the required spectral acceleration demand at the MCE level can be defined for both near-fault sites and general sites. On the other hand, there is a specific ratio between the seismic demands determined by the seismic hazard analysis at return periods of 2,500 years and 475 years for this seismic zone. Hence, the near-fault spectral acceleration demand at the MCE

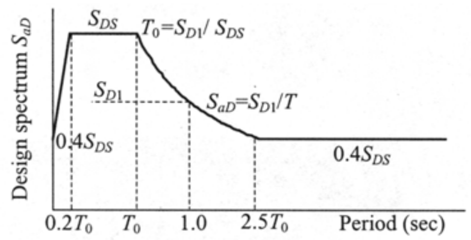

Fig. 1 Near-fault design spectral acceleration demand reduced from the MCE level

level can be reduced to the design level on the basis of the same specific ratio, and then to develop the near-fault design response spectrum for designing structures against the near-fault ground motions.

On the other hand, it has been believed that it is not feasible to design a building structure to remain elastic under intense ground motions. The seismic design has aimed to ensure that (a) the structure should not suffer any structural damage from frequently minor earthquakes, (b) the repaired structure should be usable after an infrequent earthquake of major intensity, and (c) the structure should not collapse (life safety limit state) for the safety of occupants during the largest possible earthquake at the construction site. Therefore, in addition to the development of near-fault design response spectrum, an alternative method called two-level design method is developed. As shown in Fig. 2, no near-fault effect is considered at the primary force-based design level, and the seismic demand determined by the seismic hazard analysis at a return period of 475 years is adopted to develop the design response spectrum for both the general sites and near-fault sites. However, an additional capacity checking level is requested to limit the ultimate capacity of the designed structure to exceed the maximum considered seismic demand, which is defined by the required spectral acceleration demand at the MCE level. It means that the maximum considered seismic demands should be defined by the attenuation law corresponding to the maximum potential magnitude of the fault for a near-fault site and defined by the seismic demand determined probabilistically at a return period of 2,500 years for general sites, respectively. Therefore, based on the two-level design method, the near-fault effect is reflected indeed at the ultimate capacity checking level even though it is not considered at the primary force-based design level.

In the following sections, the development of seismic design base shear, the near-fault spectral acceleration demands at both the MCE level and design level, and the ultimate capacity checking requirements which are developed in the current revised seismic design code are introduced briefly.

SEISMIC DESIGN BASE SHEAR

For the current development of seismic design code in Taiwan [7,8], the elastic seismic demand is represented by the design spectral response acceleration S_{aD} corresponding to a uniform seismic hazard level of 10% probability of exceedance within 50 years (return period of 475 years). Based on the uniform hazard analysis, the mapped design 5% damped spectral response acceleration at short periods (S_S^D) and at 1 second (S_1^D) are determined and prepared for each administration unit of village, town or city level. These spectral response acceleration parameters should be modified by site coefficients to include local site effects, and the site adjusted spectral response acceleration at short periods (S_{DS}) and at 1 second (S_{D1}) are expressed as

$$S_{DS} = F_a\, S_S^D \; ; \qquad S_{D1} = F_v\, S_1^D \tag{1}$$

where site coefficients F_a and F_v are defined in Tables 1 and 2, and they are functions of the soil type and the mapped spectral response acceleration parameters, S_S^D for F_a and S_1^D for F_v, respectively.

Based on the soil structures in the upper 30 meters below the ground surface, the site can be classified into three classes by using $\overline{V}_s$-method, $\overline{N}$-method or $\overline{s}_u$-method as shown in Table 3. The site class parameters $\overline{V}_s$ and $\overline{N}$ are defined as the averaged shear wave velocity and averaged standard penetration resistance for all

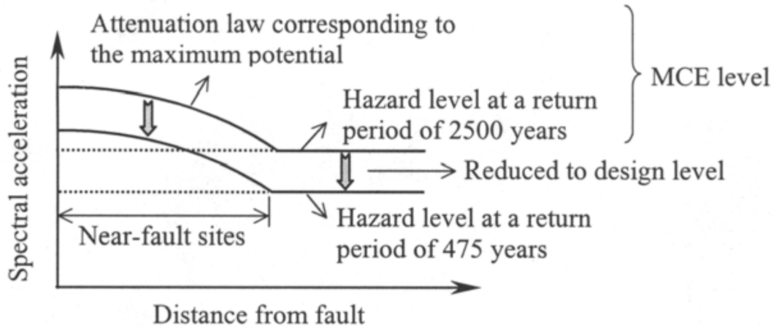

Fig. 2 Two-level design: near-fault effect is considered at the capacity checking level

Table 1 Values of site coefficients F_a

Site Class	Values of F_a				
	$S_S \le 0.5$	$S_S = 0.6$	$S_S = 0.7$	$S_S = 0.8$	$S_S \ge 0.9$
S1 (Hard Site)	1.0	1.0	1.0	1.0	1.0
S2 (Normal site)	1.2	1.2	1.1	1.1	1.0
S3 (Soft Site)	1.4	1.3	1.2	1.2	1.1

Note: S_S may be $S_S{}^D$, $S_S{}^M$, $N_A S_{S,B}^D$ or $N_A S_{S,B}^M$ for different cases, and straight- line interpolation for intermediate values of S_S is used

Table 2 Values of site coefficients F_v

Site Class	Values of F_v				
	$S_1 \le 0.3$	$S_1 = 0.35$	$S_1 = 0.4$	$S_1 = 0.45$	$S_1 \ge 0.5$
S1 (Hard Site)	1.0	1.0	1.0	1.0	1.0
S2 (Normal site)	1.5	1.4	1.3	1.3	1.2
S3 (Soft Site)	1.7	1.6	1.5	1.5	1.4

Note: S_1 may be $S_1{}^D$, $S_1{}^M$, $N_V S_{1,B}^D$ or $N_V S_{1,B}^M$ for different cases, and straight-line interpolation for intermediate values of S_S is used

Table 3 Site classification

Site Class	$\overline{V}_s$ -method	$\overline{N}$ -method	$\overline{s}_u$ -method	
	$\overline{V}_s$ (m/s)	$\overline{N}$	$\overline{N}_{ch}$	$\overline{s}_u$ (kPa)
S1 (Hard site)	$\overline{V}_s > 360$	$\overline{N} > 50$	$\overline{N}_{ch} > 50$	$\overline{s}_u > 100$
S2 (Normal site)	$180 \le \overline{V}_s \le 360$	$15 \le \overline{N} \le 50$	$15 \le \overline{N}_{ch} \le 50$	$50 \le \overline{s}_u \le 100$
S3 (Soft site)	$\overline{V}_s < 180$	$\overline{N} < 15$	$\overline{N}_{ch} < 15$	$\overline{s}_u < 50$

Note: If the $\overline{s}_u$ -method is used and the $\overline{N}_{ch}$ and $\overline{s}_u$ criteria differ, select the category with the softer soils

soil layers in the top 30m, respectively, and they are determined by

$$\overline{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}; \quad \overline{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{2}$$

where V_{si} is the shear wave velocity, N_i is the standard penetration resistance not to exceed 100 as directly measured in the field without corrections, and d_i is the thickness of any layer with $\sum_{i=1}^{n} d_i$ = 30m. On the other hand, if the $\overline{s}_u$ -method is adopted, the averaged standard penetration resistance $\overline{N}_{ch}$ for cohesionless soil layers (PI < 20) and averaged undrained shear strength $\overline{s}_u$ for cohesive soil layers (PI > 20) in the top 30m can be determined by

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} (d_i / N_i)}$$

$$\sum_{i=1}^{m} d_i = d_s \text{ (cohesionless soils)}$$

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k}(d_i / s_{ui})}$$
$$\sum_{i=1}^{k} d_i = d_c \text{ (cohesive soils)} \quad (3)$$

where d_s and d_c are the total thickness of cohesionless and cohesive soil layers in the top 30m ($d_s + d_c$ = 30m), respectively, and s_{ui} is the undrained shear strength not to exceed 250kPa.

Based on the site adjusted spectral response acceleration parameters S_{DS} and S_{D1}, the design spectral response acceleration S_{aD} can be developed by

$$S_{aD} = \begin{cases} S_{DS}(0.4 + 3\,T/T_0) & ; \; T \le 0.2\,T_0 \\ S_{DS} & ; \; 0.2\,T_0 < T \le T_0 \\ S_{D1}/T & ; \; T_0 < T \le 2.5\,T_0 \\ 0.4\,S_{DS} & ; \; T > 2.5\,T_0 \end{cases}$$
$$\text{with} \quad T_0 = \frac{S_{D1}}{S_{DS}} \quad (4)$$

where T is the structure period in the unit of second, and the shape of design response spectrum is shown in Fig. 3.

The structure system ductility capacity R for some basic types of seismic-force-resisting system can be found in the seismic design code, and further, the allowable ductility capacity R_a can be defined by

$$R_a = 1 + (R - 1)/1.5 \quad (5)$$

It implies that only two-third of the ultimate inelastic deformation capacity is permitted to be utilized. Based on the equal displacement principle between elastic and elastic-plastic systems for long period range and equal energy principle for short periods, the structure system seismic reduction factor F_u can be defined by the allowable ductility capacity R_a and structure period T as

$$F_u = \begin{cases} R_a & ; \; T \ge T_0 \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1}) \times \dfrac{T - 0.6\,T_0}{0.4\,T_0} & ; \; 0.6\,T_0 \le T \le T_0 \\ \sqrt{2R_a - 1} & ; \; 0.2\,T_0 \le T \le 0.6\,T_0 \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1) \times \dfrac{T - 0.2\,T_0}{0.2\,T_0} & ; \; T \le 0.2\,T_0 \end{cases} \quad (6)$$

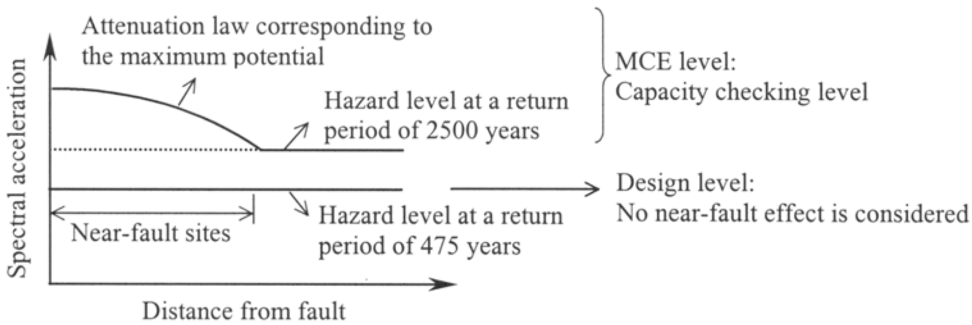

Fig. 3 Design response spectrum developed by site adjusted parameters S_{DS} and S_{D1}

As shown in Eq. (6), the structural period larger than T_0 is defined as the long period range with T_0 being the corner period of the design response spectrum as defined by Eq. (4). On the other hand, the constant acceleration range is divided into two equal parts, the structural period in the range of $0.2T_0$ to $0.6T_0$ is defined as the short period range, and the linear interpolation is defined for the other part ($0.6T_0$ to T_0) between short and long period ranges. Furthermore, the linear interpolation is also defined for structural period less than $0.2T_0$, such that the reduction factor F_u will be equal to one when the structural period becomes zero, because no ductility can be considered for a rigid body.

Finally, the seismic design base shear can be expressed as

$$V = \frac{I}{1.4\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m W \quad \text{(for Buildings)};$$

$$V = \frac{I}{1.2\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m W \quad \text{(for Bridges)} \qquad (7)$$

and

$$\left(\frac{S_{aD}}{F_u}\right)_m = \begin{cases} \dfrac{S_{aD}}{F_u} & ; \ \dfrac{S_{aD}}{F_u} \le 0.4 \\ 0.55\dfrac{S_{aD}}{F_u} + 0.18 & ; \ 0.4 < \dfrac{S_{aD}}{F_u} \le 0.9 \\ 0.75\dfrac{S_{aD}}{F_u} & ; \ \dfrac{S_{aD}}{F_u} > 0.9 \end{cases} \qquad (8)$$

herein, I is the important factor, W is the total gravity load of the structures, α_y is the first yield seismic force amplification factor that is dependent on the structure types and design method. The constant 1.4 (for buildings) or 1.2 (for bridges) means the over strength factor between the ultimate and first yield forces, and it is dependent on the redundancy of the structural system. The modified ratio of $(S_{aD}/F_u)_m$ is defined to reduce the seismic demand because the higher damping ratio (5% ~ 10%) will be caused by the soil-structure interaction for short period structures. The procedures to determine the seismic design base shear are outlined in Fig. 4.

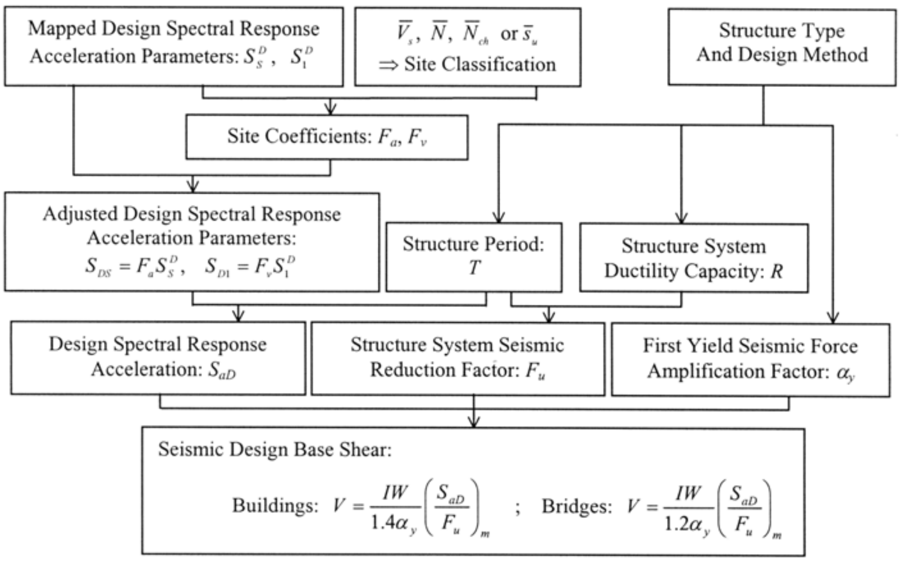

Fig. 4 Procedures to determine the seismic design base shear

NEAR-FAULT DESIGN RESPONSE SPECTRUM

To consider the effect of near-fault ground motions in seismic design, both the probabilistic analysis based on the seismic hazard analysis at a return period of 2,500 years and the deterministic analysis based on the attenuation law corresponding to the maximum potential magnitude of the fault are implemented. Based on the uniform hazard analysis at a return period of 2,500 years, the mapped spectral response acceleration parameters S_S^M and S_1^M can be determined for each administration unit near the fault of interest. Furthermore, the averaged demand $S_{S,B}^M$ and $S_{1,B}^M$ can be determined, and they are recognized as the lower limit of seismic demand in the near-fault zone at the MCE level caused by other potential sources. On the other hand, the attenuation relations $S_{S,Att}(r)$ and $S_{1,Att}(r)$ for the median 5% damped spectral acceleration demands at short periods (e.g., 0.3 second period) and at 1 second can be developed on the basis of the maximum potential magnitude of the specified active fault. For example, the Chi-Chi earthquake with a magnitude of M_L = 7.3 can be recognized as the maximum potential earthquake of Chelungpu fault. Compared with the lower limit of the spectral response acceleration parameters at the MCE level, the near-fault factors $N_A(r)$ and $N_V(r)$ can be defined by

$$N_A(r) = \frac{1.5\, S_{S,Att}\,(r)}{S_{S,B}^M}\;; \quad N_V(r) = \frac{1.5\, S_{1,Att}\,(r)}{S_{1,B}^M} \tag{9}$$

The factor of 1.5 implies the consideration of 1σ deviation of uncertainty of fault movement and the component effect (fault-normal). Therefore, the required spectral response acceleration at short periods ($S_{S,NF}^M$) and at 1 second ($S_{1,NF}^M$) for the near-fault zone at the MCE level can be defined by

$$S_{S,NF}^M(r) = N_A(r)\, S_{S,B}^M\;;\; S_{1,NF}^M(r) = N_V(r)\, S_{1,B}^M \tag{10}$$

and they are functions of the distance from the fault.

In order to determine the reduced factor from the MCE level to design level, the mapped spectral response acceleration parameters S_S^D and S_1^D for each administration within the same near-fault zone should be averaged to define the lower limit of seismic demand at the design level, and denoted by $S_{S,B}^D$ and $S_{1,B}^D$, respectively. Based on the reduced factors, which are defined by $R_s = S_{S,B}^D / S_{S,B}^M$ and $R_1 = S_{1,B}^D / S_{1,B}^M$, the required spectral response acceleration at short periods ($S_{S,NF}^D$) and at 1 second ($S_{1,NF}^D$) for the near-fault zone at the design level can be defined from Eq. (10) as

$$S_{S,NF}^D(r) = N_A(r)\, S_{S,B}^D\;;\; S_{1,NF}^D(r) = N_V(r)\, S_{1,B}^D \tag{11}$$

Similar to Eq. (1), the site-adjusted near-fault spectral response acceleration parameters S_{DS} and S_{D1} can be determined by

$$S_{DS} = F_a\, N_A\, S_{S,B}^D\;;\; S_{D1} = F_v\, N_V\, S_{1,B}^D \tag{12}$$

It is noted that the associated site coefficients F_a and F_v should be evaluated from Tables 1 and 2 on the basis of the near-fault spectral response acceleration parameters $N_A\, S_{S,B}^D$ and $N_V\, S_{1,B}^D$, respectively. Therefore, substituting Eq. (12) into Eq. (4), the required near-fault design response spectrum can be developed, and then to determine the seismic design base shear for designing structures. Because the near-fault effect is considered at the design level, it can intensify the resistance capacity of structures against near-fault ground motions directly.

TWO-LEVEL DESIGN METHOD

In addition to the development of near-fault seismic design base shear for designing structures, an alternative two-level design method is developed to consider the near-fault effect. At the primary force-based design level, only the mapped design spectral response acceleration parameters (S_S^D and S_1^D) corresponding to a return period of 475 years are adopted to determine the design base shear for both the general sites and near-fault sites. Then, an additional capacity checking level is requested to limit the ultimate capacity of the designed structure to exceed the maximum considered seismic demand defined at the MCE level.

For determining the maximum seismic demand at the checking level for general sites, the site-adjusted spectral response acceleration at short periods (S_{MS}) and at 1 second (S_{M1}) can be defined by the mapped spectral response acceleration parameters S_S^M and S_1^M at the MCE level as

$$S_{MS} = F_a\, S_S^M \; ; \quad S_{M1} = F_v\, S_1^M \tag{13}$$

herein, the associated site coefficients F_a and F_v should be evaluated from Tables 1 and 2 on the basis of S_S^M and S_1^M, respectively. For near-fault sites, the site-adjusted spectral response acceleration at short periods (S_{MS}) and at 1 second (S_{M1}) should be defined by the near-fault spectral response acceleration parameters, $N_A\, S_{S,B}^M$ and $N_V\, S_{1,B}^M$, at the MCE level as

$$S_{MS} = F_a\, N_A\, S_{S,B}^M \; ; \; S_{M1} = F_v\, N_V\, S_{1,B}^M \tag{14}$$

and the associated site coefficients F_a and F_v should be evaluated from Tables 1 and 2 on the basis of the near-fault spectral response acceleration parameters $N_A\, S_{S,B}^M$ and $N_V\, S_{1,B}^M$, respectively.

Then, the required spectral response acceleration S_{aM} at the checking level can be developed as

$$S_{aM} = \begin{cases} S_{MS}(0.4 + 3T/T_0^M) & ; \; T \le 0.2T_0^M \\ S_{MS} & ; \; 0.2T_0^M < T \le T_0^M \\ S_{M1}/T & ; \; T_0^M < T \le 2.5T_0^M \\ 0.4S_{MS} & ; \; T > 2.5T_0^M \end{cases}$$

$$\text{with} \quad T_0^M = \frac{S_{M1}}{S_{MS}} \tag{15}$$

Furthermore, at the ultimate capacity checking level, the ductility demand is allowed to reach its capacity R instead of the allowable ductility capacity R_a as defined for the primary force-based design level. Therefore, the structure system seismic reduction factor F_{uM} at the checking level can be defined by

$$F_{uM} = \begin{cases} R & ; \; T \ge T_0^M \\ \sqrt{2R-1} + (R - \sqrt{2R-1}) \times \dfrac{T - 0.6T_0^M}{0.4T_0^M} & ; \; 0.6T_0^M \le T \le T_0^M \\ \sqrt{2R-1} & ; \; 0.2T_0^M \le T \le 0.6T_0^M \\ \sqrt{2R-1} + (\sqrt{2R-1} - 1) \times \dfrac{T - 0.2T_0^M}{0.2T_0^M} & ; \; T \le 0.2T_0^M \end{cases} \tag{16}$$

The criteria for the ultimate capacity check is that the allowable lateral capacity P_a should exceed the maximum shear force demand, i.e.,

$$P_a > \left(\frac{S_{aM}}{F_{uM}}\right)_m I\,W \tag{17}$$

herein, the modified ratio $(S_{aM}/F_{uM})_m$ is defined by

$$\left(\frac{S_{aM}}{F_{uM}}\right)_m = \begin{cases} \dfrac{S_{aM}}{F_{uM}} & ; \ \dfrac{S_{aM}}{F_{uM}} \le 0.4 \\ 0.55\dfrac{S_{aM}}{F_{uM}} + 0.18 & ; \ 0.4 < \dfrac{S_{aM}}{F_{uM}} \le 0.9 \\ 0.75\dfrac{S_{aM}}{F_{uM}} & ; \ \dfrac{S_{aM}}{F_{uM}} > 0.9 \end{cases} \tag{18}$$

It is noted that the two-level design for bridges at near-fault sites is considered in the current revised code in Taiwan. The seismic demand caused by the near-fault effect at checking level, the estimation of ultimate capacity of a RC bridge pier and the checking requirements are developed in the current revised seismic design code for bridges [8]. On the other hand, because the allowable lateral capacity P_a for buildings can hardly be evaluated, the two-level design method is simplified by defining the seismic design base shear as

$$V = \max\left[\frac{I}{1.4\,\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m W\,,\ \frac{I}{1.4\,\alpha_y}\left(\frac{S_{aM}}{F_{uM}}\right)_m W\right] \tag{19}$$

and the second ultimate capacity checking process can be dropped. It is noted that, based on the current revised seismic design code for buildings, Eq. (19) should be considered for both near-fault sites and general sites.

CONCLUSIONS

Based on the uniform hazard analysis at a return period of 475 years, the mapped design 5% damped spectral response acceleration at short periods and at 1 second are prepared for the specified administration unit. Furthermore, by considering the local site effect, the site-adjusted design spectral response acceleration parameters can be defined through the site coefficients and then to develop the design spectral response acceleration. Together with the system reduction factor and the first yield amplification factor, the seismic design base shear for buildings and bridges can be well defined.

For near-fault sites, based on the seismic hazard analysis at a return period of 2,500 years and the attenuation law corresponding to the maximum potential magnitude of the fault, the required spectral response acceleration parameters for the near-fault zone at the MCE level can be defined. Then, it can be either reduced to the design level to develop the near-fault design response spectrum for designing structures against near-fault ground motions directly, or utilized to define the required spectral response acceleration demand at the checking level for the two-level design method.

The two-level design for bridges at near-fault sites is considered in the current revised code in Taiwan. The seismic demand caused by the near-fault effect at checking level, the estimation of ultimate capacity of a RC bridge pier and the checking requirements are developed in the current revised seismic design code. For buildings, based on the current revised seismic design code, the design base shear defined at the design level should be limited to be larger than that defined at the MCE level for both the near-fault sites and general sites, and the second ultimate capacity checking process can be dropped.

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