PART I

Seismic Force Requirements for Buildings in Taiwan

Extracted from:

2005 Seismic Design Code for Buildings in Taiwan

Translated by:

National Center for Research on Earthquake Engineering

Seismic Force Requirements for Buildings in Taiwan

1. Introduction

Taiwan is located in the circum-Pacific earthquake belt, and most building designs are controlled by seismic loads. Seismic design codes have to be periodically revised to reflect the latest findings from both research and practice. In 1974 Taiwan implemented, seismic force requirements (SFR) for building structures based on the format of the US Uniform Building Code. In 1982, the important factors for various building occupancy categories were further incorporated into the SFR. After the Mexico Earthquake in 1985, the importance of the fundamental vibration of the Taipei Basin was recognized and a specific acceleration response spectrum was incorporated into the SFR in 1989.

In 1997, the SFR underwent major changes. These changes include the dynamic analysis procedures using the response spectrum method, the number of seismic zones increased from 3 to 4, and the zoning factor directly represents the design peak ground acceleration associated with a hazard level of 10% chance of exceedance in 50 years (10/50 event). In addition, the force reduction factors associated with any one specific structural system follow the Newmark and Hall recommendations. Hence, the force reduction factor varies depending on the fundamental vibration periods of a given structural system. Three months after the 1999 Chi-Chi Earthquake, a change in the building codes was released that temporarily reduced the number of Taiwan seismic zones from 4 to 2.

The current version, a completely new version of the SFR, was released in 2005. In this version, the design parameters for the mapped spectral response acceleration are determined based directly on the uniform hazard analysis considering 10% probability of exceedance in 50 years (10/50 hazard or a return period of 475 years). The 5%-damped spectral response acceleration for short periods and at 1.0 second are prescribed for each municipal unit such as a village, town or city. In addition, the site-adjusted spectral response acceleration parameters for short periods and 1.0 second structures can be defined by multiplying the mapped values with the site coefficients to incorporate the local site effects. The design spectral response acceleration can then be computed on the basis of the site-adjusted spectral response acceleration parameters. Thus, it can be used to determine the design base shear.

Similar to the UBC97, after the Chi-Chi earthquake (1999) the so-called near-fault factors N_A and N_V were implemented in Taiwan in order to consider the near-fault effect. Two near-fault factors defined for the short period (acceleration control) and the long period (velocity control) domains were considered since the effects are substantially greater for longer period structures. In this new seismic building code, the values for the near-fault factors N_A and N_V are prescribed for several active faults in Taiwan.

Furthermore, four seismic micro-zones were defined for the Taipei Basin to reflect the observed basin effects due to the varied thickness of the sedimentary soil layers in these regions. The specific value of the corner period T_0 between the short and the moderate period ranges of the design response spectrum were defined for each micro-zone. Thus, applying the uniform hazard analysis, design spectral response acceleration values for structures in Taipei Basin can be determined directly from the design spectral response acceleration for short period structures as well as from the corner period T_0 prescribed for each micro-zone.

In addition to the seismic demand considered for the 10/50 hazard, the seismic demand imposed by the maximum considered earthquake (MCE) was also incorporated into the current seismic building provisions in order to avoid the collapse of buildings during an extremely large earthquake. In the current seismic building code, the MCE hazard level is defined as a seismic hazard level of 2% probability of exceedance within 50 years (2/50 hazard or a return period of 2500 years). Furthermore, in order to avoid any nonlinear demand on the structural elements during a frequently occurring small earthquake, a minimum seismic force (MSF) requirement is prescribed in the current seismic code. The final base shear for the elastic structural design is governed by the larger of those determined at the design level (using a reduced ductility capacity against the 10/50 hazard) and the MCE level (using the full system ductility against the 2/50 hazard). Nevertheless, it should never be less than the MSF requirement. For the dynamic analysis procedures, both the response spectrum method and the time history method are specified in the new seismic design code.

2. Static Analysis Procedures

2.1 Seismic Design Base Shear for General Sites

In the current seismic building code in Taiwan, 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. 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) have been tabulated for each municipal unit of village, town or city level. For the sake of simplicity, only four levels of S_S^D and S_1^D were defined for both the 10/50 and the 2/50 hazard levels as shown in Table 1.

Table 1. Values of mapped spectral response acceleration parameters

10%/50 year	S_S^D (g)	0.8	0.7	0.6	0.5
	S_1^D (g)	0.45	0.40	0.35	0.30
2%/50 year	S_S^M (g)	1.0	0.9	0.8	0.7
	S_1^M (g)	0.55	0.50	0.45	0.40

The mapped spectral response acceleration parameters must be modified using the site coefficients in order to include the local site effects. Thus, the site-adjusted spectral response accelerations at short periods (S_{DS}) and at 1.0 second (S_{D1}) are expressed as:

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

where site coefficients F_a and F_v are given in Tables 2 and 3. These coefficients 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. From the above provisions it is evident that the non-linear amplification effects of soil layers have been considered.

Based on the soil structure in the upper 30 meters below the ground surface, a given site can be classified into one of the three classes using the V_{S30}-method, as shown in Table 3. The site class parameter V_{S30} is defined as the averaged shear wave velocity for all soil layers in the top 30 meters, and is determined by:

$$V_{S30} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \left(d_i / V_{si} \right)} \tag{2}$$

where V_{si} is the shear wave velocity, and d_i is the thickness of any soil layer in the top 30 meters ($\sum_{i=1}^{n} d_i = 30\,\text{m}$). The shear wave velocity at any soil layer can be obtained from the PS logging data, or estimated by the following equations:

$$\text{for a cohesive soil layer: } V_{si} = \begin{cases} 120 q_{ui}^{0.36} & ; \quad N_i < 2 \\ 100 N_i^{1/3} & ; \quad 2 \le N_i \le 25 \end{cases} \tag{3.a}$$

$$\text{for a cohesionless soil layer: } V_{si} = 80 N_i^{1/3} \quad ; \quad 1 \le N_i \le 50 \tag{3.b}$$

where N_i is the standard penetration resistance as measured in the field without corrections, and q_{ui} is the unconfined compression strength (in kgf/cm^2).

Table 2. Values of site coefficients F_a and F_v

Site Class	Values of F_a					Values of F_v				
	S_S≤0.5	S_S =0.6	S_S =0.7	S_S =0.8	S_S ≥0.9	S_1≤0.3	S_1 =0.35	S_1=0.4	S_1=0.45	S_1 ≥0.5
Hard site	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Normal site	1.1	1.1	1.0	1.0	1.0	1.5	1.4	1.3	1.2	1.1
Soft site	1.2	1.2	1.1	1.0	1.0	1.8	1.7	1.6	1.5	1.4

Note: S_S may be S_S^D, S_S^M, $N_A S_S^D$ or $N_A S_S^M$ for different cases. Straight-line interpolation is used for the intermediate values of S_S.; S_1 may be S_1^D, S_1^M, $N_v S_1^D$ or $N_v S_1^M$ for different cases, and straight-line interpolation is used for the intermediate values of S_S and S_1.

Table 3. Site classification

Site Class	V_{S30}-method (m/s)
S1 (Hard site)	V_{S30}>270
S2 (Normal site)	180≤ V_{S30}≤270
S3 (Soft site)	V_{S30}<180

Based on the site-adjusted spectral response acceleration parameters S_{DS} and S_{D1}, the design spectral response acceleration S_{aD} for a given structure can be developed by using the following:

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

where T is the structure's fundamental period given in seconds. The shape of the design response spectrum is illustrated in Fig. 1. The fundamental period can be determined by the following approximate equations:

(1) Moment resisting frame systems not enclosed or adjoined by more rigid components that will prevent the frames from deflecting under seismic forces:

Steel moment-resisting frame: $T = 0.085h_n^{3/4}$

RC or SRC moment-resisting frame: $T = 0.07h_n^{3/4}$

(2) Eccentrically braced steel frames: $T = 0.07h_n^{3/4}$

(3) Others: $T = 0.05h_n^{3/4}$

where h_n is the height (in meters) of the building above the base. In addition, the fundamental period can also be estimated by a properly substantiated analysis. However, the estimated period shall not exceed the product of the approximate fundamental period and the coefficient for the upper limit of the calculated period (C_u=1.4).

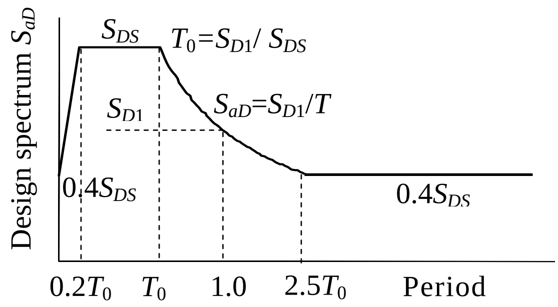

Figure 1. Design response spectrum developed from the site-adjusted parameters S_{DS} and S_{D1}

The ductility capacity R of the structural system for most basic types of seismic-force-resisting system can be found in the seismic design code. For example, the R values for a special moment steel frame and a special concentrically braced frame are 4.8 and 4.0, respectively. However, in order to control the damage level under the design base earthquake (DBE), only two-thirds of the ultimate inelastic deformational capacity of the structural system is considered in the design. Therefore, the allowable ductility capacity R_a shall be defined according to the ductility capacity R as:

$$R_a = 1 + (R-1)/1.5 \quad \text{(for general sites and near-fault sites)} \tag{5}$$

In addition, the seismic force reduction factor F_u for the structural system can be defined by the allowable ductility capacity R_a and the fundamental period T of the structure as:

$$F_u = \begin{cases} R_a & ; \quad T \geq T_0 \\ \sqrt{2R_a - 1} + \left(R_a - \sqrt{2R_a - 1}\right) \times \dfrac{T - 0.6T_0}{0.4T_0} & ; \quad 0.6T_0 \leq T \leq T_0 \\ \sqrt{2R_a - 1} & ; \quad 0.2T_0 \leq T \leq 0.6T_0 \\ \sqrt{2R_a - 1} + \left(\sqrt{2R_a - 1} - 1\right) \times \dfrac{T - 0.2T_0}{0.2T_0} & ; \quad T \leq 0.2T_0 \end{cases} \tag{6}$$

This is based on the equal displacement principle between the elastic and the EPP systems for the long period range and the equal energy principle for short periods. As shown in Eq. (6), the structural period larger than T_0 is viewed 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. The linear interpolation is also adopted for a 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. This is because there is no ductility capacity considered for a rigid body. Thus, the seismic design base shear is expressed as:

$$V = \frac{I}{1.4\alpha_y}\left(\frac{S_{aD}}{F_u}\right)_m W \tag{7}$$

and

$$\left(\frac{S_{aD}}{F_u}\right)_m = \begin{cases} \dfrac{S_{aD}}{F_u} & ; \quad \dfrac{S_{aD}}{F_u} \leq 0.3 \\ 0.52\dfrac{S_{aD}}{F_u} + 0.144 & ; \quad 0.3 < \dfrac{S_{aD}}{F_u} \leq 0.8 \\ 0.70\dfrac{S_{aD}}{F_u} & ; \quad \dfrac{S_{aD}}{F_u} > 0.8 \end{cases} \tag{8}$$

where I is the important factor, W is the total gravity dead load of the structure, α_y is defined as the first yield seismic force amplification factor that is dependent on the structure types and design method. For example, α_y=1.2 for steel structures using the allowable stress design method, and α_y=1.5 for RC structures using the strength design method. In addition, the constant 1.4 means the over-strength factor between the ultimate and the first yield force. This is somewhat dependent on the redundancy of the structural system, but is treated as a constant for the sake of simplicity. The modified ratio of $(S_{aD}/F_u)_m$ is defined to reduce the seismic demand, because a damping ratio higher than 5% can be considered due to the soil-structure interaction for short period structures. The procedures to determine the seismic design base shear for general sites are outlined in Fig. 2.

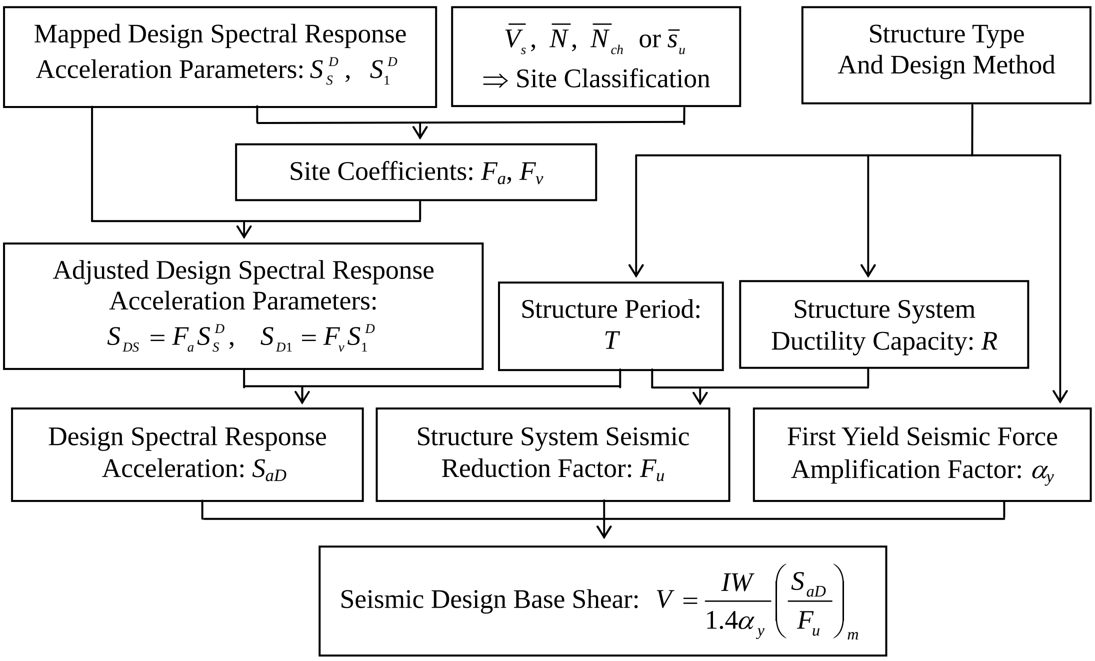

Figure 2. Procedures to determine the seismic design base shear for general sites

2.2 Seismic Design Base Shear for Near-Fault Sites

In order to take the effects of near-fault ground motions into consideration in the seismic design of structures, the near-fault factors N_A and N_V are defined for several active faults in Taiwan. Within the proximity of these specific near-fault sites, the near-fault effects should be considered at the design level to improve the seismic design force requirements of these structures against near-fault ground motions. For these specific near-fault sites, the site-adjusted spectral response acceleration parameters S_{DS} and S_{D1} can be computed from:

$$S_{DS} = F_a N_A S_S^D \quad ; \quad S_{D1} = F_v N_V S_1^D \tag{9}$$

It should be noted that the associated site coefficients F_a and F_v must be evaluated from Table 2 on the basis of the near-fault spectral response acceleration parameters $N_A S_S^D$ and $N_V S_1^D$, respectively. The near-fault factors N_A and N_V are determined on the basis of the characteristic earthquake model as well as the seismic hazard analysis for the Taiwan area. They are expressed as functions of the distance between the given building site and the near-fault. Ultimately, the site-adjusted spectral response acceleration parameters must be applied to determine the design spectral response acceleration S_{aD} using Eq. (4). Then the near-fault design base shear can be determined by the same procedure as prescribed for general sites.

2.3 Seismic Design Base Shear for the Taipei Basin

Due to the basin effects, the corner periods noted in the response spectra associated with the earthquake data observed in Taipei Basin are generally larger than 1.0 second.

This implies that the aforementioned parameters S_{DS} and S_{D1} prescribed in the design response spectrum for general sites can not be applied directly for sites in the Taipei Basin. Therefore, it is based on the parameters of C=2.5 and $C=C_v/T$ for the normalized design response spectrum within the short and moderate period ranges, respectively. Parameter C_v and the associated corner period ($T_0=C_v/2.5$) can be determined from the observed strong ground motions from each observation station within the Taipei Basin. Then, based on the contours of parameter C_v and the boundaries of the municipal units, four seismic micro-zones are defined in Taipei Basin. The representative values of corner period T_0 between short and moderate period ranges of the design response spectrum are shown in Table 4. In addition, utilizing the uniform hazard analysis, the design spectral response acceleration S_{aD} for a given site can be developed directly from the design spectral response acceleration at short periods S_{DS} (S_{DS}=0.6g) as well as the corner period T_0 for each seismic micro-zone in Taipei Basin, and can be expressed as:

$$S_{aD} = \begin{cases} S_{DS}\left(0.4 + 3T/T_0\right) & ; \quad T \le 0.2T_0 \\ S_{DS} & ; \quad 0.2T_0 < T \le T_0 \\ S_{DS}T_0/T & ; \quad T_0 < T \le 2.5T_0 \\ 0.4S_{DS} & ; \quad T > 2.5T_0 \end{cases} \tag{10}$$

The distribution of the four micro-zones and the shapes of the corresponding design response spectrum in Taipei Basin are shown in Fig. 3. It should be noted that the distribution of the four micro-zones is in accordance with the basin shape and reflects the thickness of the sedimentary soil layers in the basin.

Table 4. Representative values of the corner period for each micro-zone in Taipei Basin

Micro-zone	Taipei Zone 1	Taipei Zone 2	Taipei Zone 3	Taipei Zone 4
Range of C_v	3.6 – 4.6	2.8 – 3.6	2.2 – 2.8	1.5 – 2.2
T_0 or T_0^M (sec.)	1.60	1.30	1.05	0.85

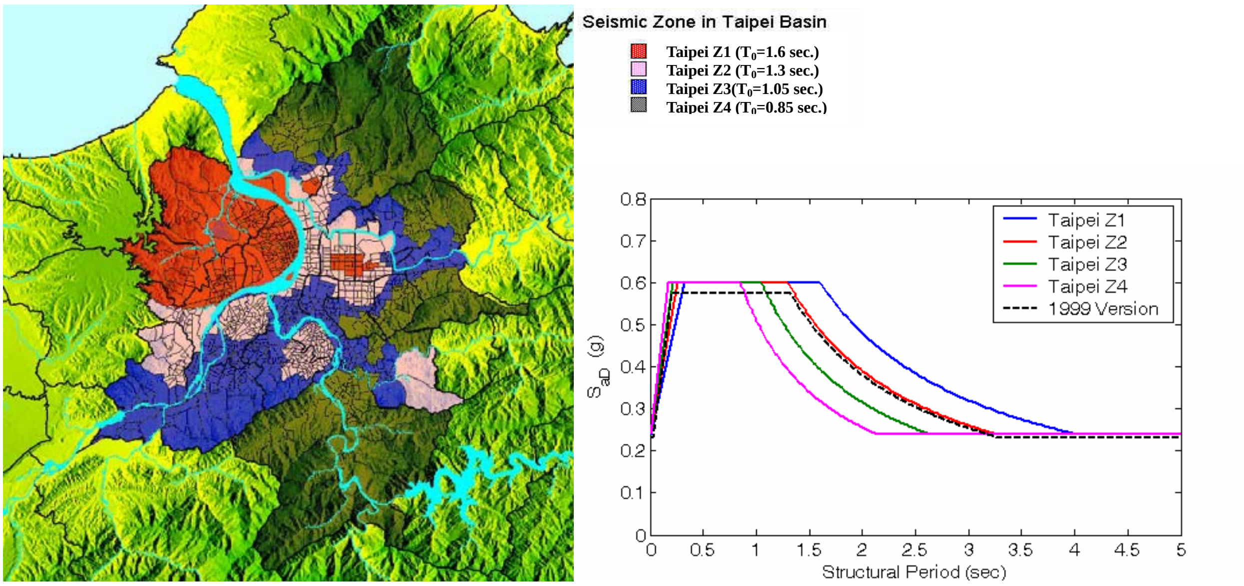

Figure 3. Distribution of the micro-zones and the design response spectrum for each micro-zone in Taipei Basin

Due to the basin effects, the duration that the ground shakes will be longer in the Taipei Basin than in any other region. Accordingly, the number of the cyclic loads imposed on the structures is likely to be greater during an earthquake. Therefore, only one-half (not two-third as suggested in Eq. 5) of the ultimate inelastic deformation capacity has been incorporated into the computation of the seismic force reduction factors for buildings located in Taipei Basin. That is, the allowable ductility capacity R_a for a given site within Taipei Basin is:

$$R_a = 1 + (R - 1)/2.0 \quad \text{(for Taipei Basin)} \tag{11}$$

Therefore, the design base-shear for any given site within the Taipei Basin can be determined using the same procedures prescribed for general sites.

2.4 Seismic Demands for MCE Hazard Level and Minimum Force Requirement

In order to avoid the collapse of a building during an extremely large earthquake, the seismic demand during a maximum considered earthquake (MCE) has been taken into consideration in the current code. For general sites, the site-adjusted spectral response acceleration at short periods (S_{MS}) and at 1.0 second (S_{M1}) has been defined using the mapped spectral response acceleration parameters S_S^M and S_1^M at the MCE level as

$$S_{MS} = F_a S_S^M \quad ; \quad S_{M1} = F_v S_1^M \tag{12}$$

In which the mapped spectral response acceleration parameters S_S^M and S_1^M at the MCE level are determined from the seismic hazard level of 2% probability of exceedance within 50 years. Similar to the design level (10/50 hazard level), only four levels of S_S^M and S_1^M have been implemented as given in Table 1.

For the near-fault sites, the site-adjusted spectral response acceleration parameters S_{MS} and S_{M1} are prescribed as:

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

The site coefficients F_a and F_v in Eqs. (12) and (13) must be evaluated from Table 2 on the basis of the mapped spectral response acceleration parameters S_S^M and S_1^M, and the near-fault spectral response acceleration parameters $N_A S_S^M$ and $N_V S_1^M$, respectively. Then, the required spectral response acceleration S_{aM} for the general sites and the near-fault sites at the MCE level can be computed from:

$$S_{aM} = \begin{cases} S_{MS}\left(0.4 + 3T/T_0^M\right) & ; \quad T \le 0.2T_0^M \\ S_{MS} & ; \quad 0.2T_0^M < T \le T_0^M \\ S_{M1}/T & ; \quad T_0^M < T \le 2.5T_0^M \\ 0.4S_{MS} & ; \quad T > 2.5T_0^M \end{cases} \quad \text{with} \quad T_0^M = \frac{S_{M1}}{S_{MS}} \tag{14}$$

At the same time, the spectral response acceleration S_{aM} for Taipei Basin at the MCE level can be computed using the spectral response acceleration at short periods S_{MS} (S_{MS}=0.8g) as well as the corner period T_0^M (defined in Table 4) for each seismic micro-zone in Taipei Basin. This is expressed as:

$$S_{aM} = \begin{cases} S_{MS}\left(0.4 + 3T/T_0^M\right) & ; \quad T \le 0.2T_0^M \\ S_{MS} & ; \quad 0.2T_0^M < T \le T_0^M \\ S_{MS}T_0^M/T & ; \quad T_0^M < T \le 2.5T_0^M \\ 0.4S_{MS} & ; \quad T > 2.5T_0^M \end{cases} \tag{15}$$

In addition, at the MCE hazard level, the system ductility demand is permitted to reach full capacity R, instead of the allowable ductility capacity R_a as prescribed for the design base earthquake (10/50 hazard level). Therefore, the seismic force reduction factor F_{uM} of the structural system at the MCE level is defined as:

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

Thus, the required base shear demand at the MCE level is defined as:

$$V_M=\frac{I}{1.4\alpha_y}\left(\frac{S_{aM}}{F_{uM}}\right)_m W \tag{17}$$

and

$$\left(\frac{S_{aM}}{F_{uM}}\right)_m=\begin{cases} \dfrac{S_{aM}}{F_{uM}} & ; \quad \dfrac{S_{aM}}{F_{uM}}\leq 0.3 \\ 0.52\dfrac{S_{aM}}{F_{uM}}+0.144 & ; \quad 0.3<\dfrac{S_{aM}}{F_{uM}}\leq 0.8 \\ 0.70\dfrac{S_{aM}}{F_{uM}} & ; \quad \dfrac{S_{aM}}{F_{uM}}>0.8 \end{cases} \tag{18}$$

Furthermore, in order to avoid any nonlinear demand on the structural elements during a frequently occurring small earthquake, a minimum seismic force (MSF) requirement is prescribed as well in the current seismic code. The corresponding base shear demand is defined as:

$$V^*=\begin{cases} \dfrac{IF_u}{4.2\alpha_y}\left(\dfrac{S_{aD}}{F_u}\right)_m W & ;\text{(for general sites and near-fault sites)} \\ \dfrac{IF_u}{3.5\alpha_y}\left(\dfrac{S_{aD}}{F_u}\right)_m W & ;\text{(for Taipei Basin)} \end{cases} \tag{19}$$

It should be noted that no near-fault effects are considered for the frequently occurring small earthquakes, and hence, the near-fault factors are defined as N_A=N_V=1.0 for the near-fault sites.

The final base shear for the elastic structural design is governed by the larger of the base shears determined at the design level (using a reduced ductility capacity against the 10/50 hazard) and the MCE level (using the full system ductility against the 2/50 hazard). Nevertheless, it should never be less than the MSF requirement. In other words, the required base shear to be used for the structural design is defined as:

$$V_D = \max\left[V, \quad V_M, \quad V^*\right] \tag{20}$$

2.5 Other Requirements

2.5.1 Distribution of Seismic Force

The vertical distribution of the determined design base-shear V_D is specified as follows. The lateral forces applied at the roof (F_t) and applied at any floor level (F_x) shall be determined in accordance with Eq. (21):

$$F_t = 0.07TV_D \quad ; \quad F_x = \frac{(V_D - F_t)W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{21}$$

where W_x is the portion of the total building weight located on or assigned to floor level x, and h_x is the height from the base to floor level x. Furthermore, the seismic forces at each floor level of the building calculated using Eq. (21) shall be distributed according to the distribution of mass at that floor level.

2.5.2 Accidental Torsional Moments

The design must include the torsional moment resulting from the location of the masses. In addition, the design shall also include the accidental torsional moments caused by an assumed displacement of the mass each way from its actual location by a distance equal to 5% of the dimension of the building perpendicular to the direction of the applied forces. Moreover, the effects of torsional irregularity must be taken into consideration by multiplying the sum of the torsional moment plus the accidental torsional moment at each level by a torsional amplification factor A_x, which is determined by

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2 \tag{22}$$

where, δ_{max} is the maximum displacement at level x, and δ_{avg} is the average of the displacement at the extreme points of the building at level x. Furthermore, it should be noted that the torsional amplification factor is not required to exceed 3.0.

2.5.3 Overturning Moments

The building shall be designed to resist overturning effects caused by the seismic forces. The overturning moments at level x shall be determined by

$$M_x = \tau \sum_{i=x}^{n} F_i \left(h_i - h_x \right) \quad \text{with} \quad \tau = \begin{cases} 1.0 & ; \quad n - x \le 10 \\ 1.0 - 0.02\left(n - x - 10 \right) & ; \quad 10 < n - x \le 20 \\ 0.8 & ; \quad n - x > 20 \end{cases} \tag{23}$$

where F_i is the seismic force as determined by Eq. (21) for level i, while h_i and h_x are the height from the base to level i or x, respectively. The variable τ represents the overturning moment reduction factor, and it should be evaluated on the basis of floor level x.

2.5.4 Drift Limits and Building Separation

The associated story drift ratio at each floor shall be determined under the following base shear:

$$V_{drift} = \frac{IF_u}{4.2}\left(\frac{S_{aD}}{F_u}\right)_m W \tag{24}$$

The story drift ratio is defined as the ratio of the difference of deflections at the top and bottom of the story under consideration divided by the story height. The story drift ratio at each floor shall not exceed 0.005.

In addition, buildings shall be adequately separated from the adjacent structures to prevent pounding during an earthquake. Pounding may be presumed not to occur wherever buildings are separated by a distance greater than or equal to $0.6\times1.4\alpha_y R_a$ times the displacement caused by the determined seismic design base shear (10/50 hazard level). The factor 0.6 is used because of the low probability that two adjacent buildings will move in the opposite directions and reach the maximum displacement simultaneously.

2.5.5 Vertical Seismic Force

The effect of the vertical response of a building to earthquake ground motion must be taken into consideration, especially for the cantilevered and pre-stressed elements and components of a structure. Based on the current seismic design code, the vertical design spectral response acceleration $S_{aD,V}$ shall be determined from the horizontal design spectral response acceleration S_{aD} by

$$S_{aD,V} = \begin{cases} S_{aD}/2 & ;\text{(for general sites and Taipei Basin)} \\ 2S_{aD}/3 & ;\text{(for near-fault sites)} \end{cases} \tag{25}$$

3. Dynamic Analysis Procedures

3.1 Scope

Buildings with any one of the following conditions shall be designed by following the dynamic analysis procedures:

(1) The building is 50m high or higher, or has more than 15 stories.

(2) The building is higher than 20m or has more than 5 stories, and it has vertical mass, stiffness or configuration irregularities, or it has torsional irregularity in any one of the stories.

(3) The building is higher than 20m or has more than 5 stories, and its structural system is non-uniform throughout its height.

For the dynamic analysis procedures, both the response spectrum method and the time history method are specified in the current version of the seismic design code.

3.2 Response Spectrum Method

When the response spectrum method is used, peak modal responses of sufficient modes have to be calculated in order to capture at least 90% of the participating mass of the building in each of the two orthogonal principal horizontal directions of the building.

Based on the modal period T_m of the m^{th} mode of the structure, the corresponding modal spectral response acceleration S_{aD}^m can be developed for general sites and near-fault sites as follows.

$$S_{aD}^m = \begin{cases} S_{DS}\left[0.4+\left(\dfrac{1}{B_S}-0.4\right)\dfrac{T_m}{0.2T_{0m}}\right] & ;\quad T_m \le 0.2T_{0m} \\ S_{DS}/B_S & ;\quad 0.2T_{0m} < T_m \le T_{0m} \\ S_{D1}/(B_1 T_m) & ;\quad T_{0m} < T_m \le 2.5T_{0m} \\ 0.4S_{DS}/B_S & ;\quad T_m > 2.5T_{0m} \end{cases} \tag{26}$$

and the corner period T_{0m} is determined by:

$$T_{0m} = \frac{S_{D1}B_S}{S_{DS}B_1} \tag{27}$$

Herein, the site-adjusted spectral response acceleration at short periods (S_{DS}) and at 1.0 second (S_{D1}) are determined from Eq. (1). The damping coefficients B_S and B_1 are defined in Table 5, expressed in terms of the effective modal damping ratio. We then find that B_S=B_1=1.0 if the damping ratio is equal to 5%, and Eqs. (26) and (27) will be reduced to Eq. (4) for this special case.

On the other hand, the modal spectral response acceleration S_{aD}^m for Taipei Basin can be developed as:

$$S_{aD}^m = \begin{cases} S_{DS}\left[0.4+\left(\dfrac{1}{B_S}-0.4\right)\dfrac{T_m}{0.2T_{0m}}\right] & ;\quad T_m \le 0.2T_{0m} \\ S_{DS}/B_S & ;\quad 0.2T_{0m} < T_m \le T_{0m} \\ T_0 S_{DS}/(B_1 T_m) & ;\quad T_{0m} < T_m \le 2.5T_{0m} \\ 0.4S_{DS}/B_S & ;\quad T_m > 2.5T_{0m} \end{cases} \tag{28}$$

and

$$T_{0m} = \frac{T_0 B_S}{B_1} \tag{29}$$

Herein, T_0 is the representative corner period (5% damping) for each micro-zone in the Taipei Basin as specified in Table 4.

Peak member forces, story displacements, story drifts, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule.

Table 5. Damping Coefficients B_s and B_1

Effective modal damping ratio ξ (%)	B_s	B_1
≤2	0.80	0.80
5	1.00	1.00
10	1.33	1.25
20	1.60	1.50
30	1.79	1.63
40	1.87	1.70
≥50	1.93	1.75

Note: The damping coefficient should be based on linear interpolation for effective modal damping ratios other than those given.

3.3 Time History Analysis

When the time history method is applied, building responses can be computed at discrete time steps using synthetic time histories as the base motion input. No fewer than three time history analyses shall be performed. Each input ground motion shall have magnitude, fault distance, and source mechanisms that are consistent with those that control the design earthquake ground motion. Furthermore, the input ground motion shall be compatible with the design response spectrum. The synthetic time history shall be scaled such that the associated 5%-damped spectral response acceleration for each period between $0.2T$ and $1.5T$ (where T is the fundamental period of the building) does not fall below 90% of the value specified by the design response spectrum. The average value in this period range shall be larger than or equal to the value averaged from the design response spectrum as prescribed by the code. Response parameters shall be calculated from each time history analysis, and the maximum value of each response parameter may be used for the design.

3.4 Adjustment by Base Shear

The force and the deformation determined by the dynamic analysis procedures shall be adjusted according to the base shear as specified below:

(1) For irregular buildings, the base shear determined by the dynamic analysis shall be adjusted to 100% of the required base shear V_D as defined by Eq. (20).

(2) For regular buildings, the base shear determined by the dynamic analysis shall be adjusted to 90% of the required base shear V_D as defined by Eq. (20).

(3) For irregular and regular buildings, if the base shear determined by the dynamic analysis exceeds 100 % and 90% of the required base shear V_D, respectively, the response determined by the dynamic analysis shall be used for the design without any

adjustment.

PART II

Seismic Force Requirements for Bridges in Taiwan

Extracted from

2000 Seismic Design Code for Bridges in Taiwan

Translated by

National Center for Research on Earthquake Engineering

Seismic Force Requirements for Bridges in Taiwan

1. Static Analysis Method

1.1 Scope of Application

For regular shaped bridges, which do not require the use of dynamic analysis, the calculation for the seismic forces are provided following this section and are performed by a static approach.

1.2 Minimum Design Horizontal Seismic Forces

For each design unit of a bridge, the minimum design horizontal seismic force V must be calculated separately for the car-driving (longitudinal) direction and the lateral (transverse) direction perpendicular to it, according to the following formulas:

$$V = \frac{ZICW}{1.2\alpha_y F_u} \tag{1}$$

where

$$\frac{C}{F_u} \le \begin{cases} 1.2(R^* = 2.0) \\ 1.1(R^* = 2.0) \\ 1.0(R^* = 5.0) \end{cases} \tag{2}$$

due to the restriction set by formula (2), $\frac{C}{F_u}$ can be referred to as $\left(\frac{C}{F}\right)_m$, and formula (1) can be rewritten as

$$V = \frac{ZI}{1.2\alpha_y}\left(\frac{C}{F_u}\right)_m W \tag{3}$$

where:

C: Coefficient of site-dependent acceleration response spectrum which is normalized to a specific acceleration of 1.0g.

W: Total dead load of the bridge design unit, including the weight of the superstructure and the piers.

Z: Seismic zone dependent horizontal acceleration factor.

I: Importance factor.

α_y : Seismic force amplification factor at the initial yielding.

F_u: Seismic force reduction factor for different structural systems.

R^*: Characteristic factor for different structural systems.

$\left(\frac{C}{F_u}\right)_m$: Modified acceleration response spectrum factor

1.3 Seismic Zone Dependent Horizontal Acceleration Factor Z

The seismic zone dependent horizontal acceleration factor Z represents the ratio of 475-year recurrence seismic ground acceleration at the seismic zone considered to the gravitational acceleration g.

Taiwan is divided into seismic zones A and B as shown in Fig.1. Their corresponding acceleration factors are 0.33 and 0.23, respectively.

Kinmen and Masu do not belong to any of the seismic zones mentioned above. However, their seismic zone dependent horizontal acceleration factor Z can be taken as 0.23.

1.4 Importance Factor I

I = 1.2 for all essential bridges which must maintain their function during a seismic disaster; I = 1.0 for all other bridges.

1.5 Dimensionless Site Dependent Acceleration Response Spectrum Coefficient C

Dimensionless site dependent acceleration response spectrum coefficient C varies with the period of vibration T of the bridge being considered and the Soil profile (as shown in Table 1).

Table 1 Horizontal Spectral Acceleration Coefficients C vs. T (sec)

Soil Profile Classification	Very Short Periods	Relatively Short Periods	Short Periods	Intermediate Periods	Long Periods
Type I	$T \leqq 0.03$ $C = 1.0$	$0.03 \leqq T \leqq 0.15$ $C=12.5T+0.625$	$0.15 \leqq T \leqq 0.333$ $C=2.5$	$0.333 \leqq T \leqq 1.315$ $C=\frac{1.2}{T^{2/3}}$	$T \geqq 1.315$ $C=1.0$
Type II	$T \leqq 0.03$ $C = 1.0$	$0.03 \leqq T \leqq 0.15$ $C=12.5T+0.625$	$0.15 \leqq T \leqq 0.465$ $C=2.5$	$0.465 \leqq T \leqq 1.837$ $C=\frac{1.5}{T^{2/3}}$	$T \geqq 1.837$ $C=1.0$
Type III	$T \leqq 0.03$ $C = 1.0$	$0.03 \leqq T \leqq 0.2$ $C=8.824T+0.7352$	$0.2 \leqq T \leqq 0.611$ $C=2.5$	$0.611 \leqq T \leqq 2.415$ $C=\frac{1.8}{T^{2/3}}$	$T \geqq 2.415$ $C=1.0$
Taipei Basin	$T \leqq 0.03$ $C = 1.0$	$0.03 \leqq T \leqq 0.2$ $C=8.824T+0.7352$	$0.2 \leqq T \leqq 1.32$ $C=2.5$	$1.32 \leqq T \leqq 3.3$ $C=\frac{3.3}{T}$	$T \geqq 3.3$ $C=1.0$

The procedure to calculate the fundamental vibration period of a bridge is as follows.

(1) Apply load $w(x)$ on the bridge along the longitudinal or transverse direction. $w(x)$

is the weight per unit length of the dead load of the bridge superstructure and tributary substructure [Force/Length].Then calculate the deflection $U(x)$ over the length of the bridge along the longitudinal or transverse direction.

(2) Calculate β and γ according to the following formulas:

$$\beta = \int w(x)U(x)dx \tag{4}$$

$$\gamma = \int w(x)U^2(x)dx \tag{5}$$

The computed factors, β, γ, are in units of (force × length), and (force × length^2), respectively.

(3) Calculate the fundamental vibration period, T, of the bridge using the expression:

$$T = 2\pi\sqrt{\delta} \tag{6}$$

where $\delta = \gamma/\beta$

The type of soil profile is determined by the period of the ground stratum T_G: (1)$T_G \leqq 0.2$sec : Soil Profile Type I is a profile with stiff and hard soil; (2) 0.2sec $< T_G \leqq 0.6$sec : Soil Profile Type II is a profile with medium soil; (3) $T_G > 0.6$sec : Soil Profile Type III is a profile with soft and weak soil.

The period of ground stratum T_G shall be calculated by the following formula:

$$T_G = 4\sum_{i=1}^{n} \frac{H_i}{V_{si}} \tag{7}$$

where H_i (m) is the thickness of the i-th soil stratum, V_{si} (m/sec) is the average shear elastic wave velocity of the i-th soil stratum. V_{si} can be calculated according to the following empirical formula:

For a clayey soil stratum : $V_{si} = 100N_i^{1/3}$, $(1 < N_i < 25)$ (8)

For a sandy soil stratum : $V_{si} = 80N_i^{1/3}$, $(1 < N_i < 50)$ (9)

In formulas (8) and (9), N_i is the average blow number N obtained from the standard penetration test for the i-th layer of soil stratum, where a total of n layer of soil stratum is supported on the bearing stratum. The bearing stratum can be either a clayey soil with a value N greater than 25, or a sandy soil with a value N greater than 50, or a soil stratum characterized by a shear wave velocity greater than 300 m/sec.

For the Taipei basin district, the value of T_G need not be calculated, because its normalized acceleration response spectrum coefficient can be determined directly according to Table 1.

1.6 Seismic Force Amplification Factor at Initial Yielding α_y

Factor α_y accounts for the initiation of the first yield section to occur after the design seismic ground acceleration is amplified by α_y times. The magnitude of α_y may vary with the design method adopted. For instance, α_y =1.70 for steel bridges using the Allowable Stress Design Method, and α_y =1.65 for RC bridges using Stress Design Method. For other cases, the value of α_y should be determined by more rigorous methods of analysis.

1.7 Seismic Force Reduction Factor of Structural System F_u

Factor F_u is related to the ductility capacity of the structural system R, the fundamental vibration period T and the soil profile type, in which the ductility capacity of the structural system R is determined by the characteristic factor of the structural system R^*:

$$R = \frac{R^*}{1.2} \tag{10}$$

where R^* varies with the type of bridge substructure system, as listed in Table 2.

Table 2 Characteristic factor of the structural system R^*

Classification	Substructure[1]	R^*
1	Wall-type Piers[2]	2
2	Single-Column Piers	3
3	Multi-Column Piers	5
4	Reinforced Concrete Pile Bents	
	a. Vertical piles only	3
	b. One or more batter piles	2
5	Steel or Composite Steel and Concrete Pile Bents	
	a. Vertical piles only	5
	b. One or more batter piles	3

[1] The R-factor is to be used for both orthogonal axes of the substructure.

[2] A wall-type pier may be designed as a column in the weak direction of the pier provided all provisions for columns specified in this specification are followed, and the *R*-factor for a single column may then be used

The allowable capacity of ductility R_a is related to the ductility capacity of structural system R based on the following formula:

$$R_a = 1 + \frac{R-1}{2.0} \tag{11}$$

For a π-Type frame bridge, with a column base that is connected to the foundation with a pinned connection and a column top that is connected to the superstructure with a fixed connection, the characteristic factor of structural system R^* along the longitudinal direction is the same as that of a single-column pier, i.e., 3.0; while if the column base is connected to the foundation with a fixed connection, then R^* = 5.0. As for the transverse direction, R^*= 2.0.

For a multi-span rigidly connected continuous girder bridge, if the pier top and base are equipped with proper lateral reinforcement for confinement, then the characteristic factor R^* which along the longitudinal direction can be taken as 5.0, which is the same as that of a multi-column pier.

For an arch-type bridge with either rigid or pinned supports, the characteristic factor R^* along the longitudinal direction can be taken as 3.0. For the transverse direction, the characteristic factor R^* is given as 2.0

For bridges with hollow circular or rectangular columns, if properly anchored transverse reinforcement is provided to confine the compressed concrete within the core of the columns, then the characteristic factors R^* of bridges with single-column piers and multi-column piers can be taken as 3.0 and 5.0, respectively.

The values of F_u for different types of soil profiles can be expressed by R_a and T according to the following formulas:

(1) Type I ground stratum (firm soil)

$$F_u = \begin{cases} R_a & ,\mathrm{T} \geq 0.333\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.242)}{0.091} & ,0.242\mathrm{sec} \leq \mathrm{T} \leq 0.333\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.15\mathrm{sec} \leq \mathrm{T} \leq 0.242\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.15)}{0.12} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.15\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \tag{12}$$

(2) Type II ground stratum (medium soil)

$$F_u = \begin{cases} R_a & ,\mathrm{T} \geq 0.465\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.308)}{0.157} & ,0.308\mathrm{sec} \leq \mathrm{T} \leq 0.465\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.15\mathrm{sec} \leq \mathrm{T} \leq 0.308\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.15)}{0.12} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.15\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \tag{13}$$

(3) Type III ground stratum (soft soil)

$$F_u = \begin{cases} R_a & ,\mathrm{T} \geq 0.611\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T-0.406)}{0.205} & ,0.406\mathrm{sec} \leq \mathrm{T} \leq 0.611\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.2\mathrm{sec} \leq \mathrm{T} \leq 0.406\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T-0.2)}{0.17} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.2\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \tag{14}$$

(4) District of Taipei Basin

$$F_u = \begin{cases} R_a & ,\mathrm{T} \geq 1.32\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T-0.76)}{0.56} & ,0.76\mathrm{sec} \leq \mathrm{T} \leq 1.32\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.2\mathrm{sec} \leq \mathrm{T} \leq 0.76\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T-0.2)}{0.17} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.2\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \tag{15}$$

1.8 Distribution of Seismic Forces

The design seismic forces $p_e(x)$ applied to the bridge at point x can be calculated according to the following formula:

$$p_e(x) = F \cdot w(x)U(x) \tag{16}$$

where

$$F = \frac{V}{\int w(x)U(x)dx} = \frac{V}{\beta} \tag{17}$$

By applying $p_e(x)$ to the bridge at point x and then proceeding with the structural analysis, the member forces and deflections, which are the basis for the structural design, can be calculated.

1.9 Design Seismic Forces for Intermediate Earthquake

In order to prevent bridges with high ductility from yielding during an intermediate earthquake, the total design horizontal seismic force should not be less than V^*,

$$V^* = \frac{ZIF_u}{3.0\alpha_y}\left(\frac{C}{F_u}\right)_m W \tag{18}$$

1.10 Simulation of Bridge Structure

When performing a static analysis, the bridge structure should be modeled realistically to ensure that the simulations of geometrical shape, weight distribution, member section properties and soil-structure interaction effect are accurate.

1.11 Seismic Forces Transmitted by Roller

If the connection between superstructure and substructure along which the horizontal seismic forces are applied is a roller, then the static friction forces of the roller supports will be transmitted to the substructure in the form of lateral loads. The static friction forces can be obtained by multiplying the dead load reaction forces on the support with the friction coefficient of the roller. However, the static friction forces need not be greater than the lateral seismic forces that a hinge support must withstand if the support is assumed to be a hinge.

1.12 Design Vertical Seismic Forces

For a bridge vibration unit, the total vertical seismic force of the design for the superstructure V_v can be calculated according to the following formula:

$$V_v = \frac{Z_v I C_v W}{1.2\alpha_y F_{uv}} \tag{19}$$

where,

$$\frac{C_v}{F_{uv}} \leq \begin{cases} 1.2, & R^* = 2.0 \\ 1.1, & R^* = 3.0 \end{cases} \tag{20}$$

Z_v: Seismic zone dependent vertical acceleration factor.

For seismic zone A: $Z_v = \frac{2}{3} Z$

For seismic zone B: $Z_v = \frac{1}{3} Z$

C_v: Coefficient of site dependent vertical acceleration response spectrum which is normalized to a specific acceleration of 1.0g (as listed in Table3).

F_{uv}: Vertical seismic force reduction factor for different structural systems.

Table 3 Vertical Spectral Acceleration Coefficients C_v vs. T (sec)

Soil Classification	Very Short Periods	Relatively Short Periods	Short Periods	Intermediate Periods	Long Periods
Type I	T≦0.03 C = 1.0	0.03≦T≦0.1 C=25T+0. 25	0.1≦T≦0.288 C=2.75	0.288≦T≦1.139 C= $\frac{1.2}{T^{2/3}}$	T≧1.139 C=1.1
Type II	T≦0.03 C = 1.0	0.03≦T≦0.1 C=25T+0.25	0.1≦T≦0.403 C=2.75	0.403≦T≦1.592 C= $\frac{1.5}{T^{2/3}}$	T≧1.592 C=1.1
Type III	T≦0.03 C = 1.0	0.03≦T≦0.1 C=25T+0.25	0.1≦T≦0.530 C=2.75	0.530≦T≦2.093 C= $\frac{1.8}{T^{2/3}}$	T≧2.093 C=1.1
Taipei Basin	T≦0.03 C = 1.0	0.03≦T≦0.1 C=21.43T+0.357	0.1≦T≦1.32 C=2.5	1.32≦T≦3.3 C= $\frac{3.3}{T}$	T≧3.3 C=1.0

The values of F_{uv} for different types of soil profile can be expressed by R_a and T according to the following formulas:

(1) Type I ground stratum (firm soil)

$$F_{uv} = \begin{cases} R_a & ,\mathrm{T} \geq 0.288\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.194)}{0.094} & ,0.194\mathrm{sec} \leq \mathrm{T} \leq 0.288\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.1\mathrm{sec} \leq \mathrm{T} \leq 0.194\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.1)}{0.07} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.1\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \qquad (21)$$

(2) Type II ground stratum (medium soil)

$$F_{uv} = \begin{cases} R_a & ,\mathrm{T} \geq 0.403\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.252)}{0.151} & ,0.252\mathrm{sec} \leq \mathrm{T} \leq 0.403\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.1\mathrm{sec} \leq \mathrm{T} \leq 0.252\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.1)}{0.07} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.1\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \quad (22)$$

(3) Type III ground stratum (soft soil)

$$F_{uv} = \begin{cases} R_a & ,\mathrm{T} \geq 0.53\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.315)}{0.215} & ,0.315\mathrm{sec} \leq \mathrm{T} \leq 0.53\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.1\mathrm{sec} \leq \mathrm{T} \leq 0.315\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.1)}{0.07} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.1\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \quad (23)$$

(4) District of Taipei Basin

$$F_{uv} = \begin{cases} R_a & ,\mathrm{T} \geq 1.32\mathrm{sec} \\ \sqrt{2R_a - 1} + (R_a - \sqrt{2R_a - 1})\dfrac{(T - 0.71)}{0.61} & ,0.71\mathrm{sec} \leq \mathrm{T} \leq 1.32\mathrm{sec} \\ \sqrt{2R_a - 1} & ,0.1\mathrm{sec} \leq \mathrm{T} \leq 0.71\mathrm{sec} \\ \sqrt{2R_a - 1} + (\sqrt{2R_a - 1} - 1)\dfrac{(T - 0.1)}{0.07} & ,0.03\mathrm{sec} \leq \mathrm{T} \leq 0.1\mathrm{sec} \\ 1.0 & ,\mathrm{T} \leq 0.03\mathrm{sec} \end{cases} \quad (24)$$

If a prestressed concrete girder is equipped with appropriate longitudinal reinforcements and transverse reinforcements for confinement, the corresponding characteristic factor R^* can be taken as 3.0 for calculating the values of F_{uv}. However, if the prestressed concrete girder is equipped with appropriate longitudinal reinforcements but not transverse reinforcements for confinement, the corresponding characteristic factor of the structural system R^* becomes 2.0.

The vertical seismic forces P_i^v of the design which piers and pile caps must carry can be calculated according to the following formula, and shall be applied to each node of piers and pile caps.

$$P_i^v = Z_v I W_i \tag{25}$$

where W_i is the weight of the i-th node.

1.13 Combination of Two Perpendicular Horizontal Seismic Forces

A combination of orthogonal seismic forces is used to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. The member elastic seismic forces (including moments, axial forces and shear forces) as a result from the analyses in these two perpendicular directions shall be combined to form two load combination cases as follows:

(1) Load Combination Case 1:

The seismic forces on each of the principal axes of a member shall be calculated by adding 100 percent of the absolute value of the member elastic seismic forces resulting from the analysis in one of the perpendicular (longitudinal) directions to 30 percent of the absolute value of the corresponding member elastic seismic forces resulting from the analysis in the second perpendicular direction (transverse).

(2) Load Combination Case 2:

Seismic forces and moments on each of the principal axes of a member shall be calculated by adding 100 percent of the absolute value of the member elastic seismic forces resulting from the analysis in the second perpendicular (transverse) direction to 30 percent of the absolute value of the corresponding member elastic seismic forces resulting from the analysis in the first perpendicular direction (longitudinal).

Exception: When the foundation and/or column connection forces are determined from plastic hinging of the column, the resulting forces need not be combined as specified in this section. For a wall-type pier, this exception only applies for the weak direction of the pier when forces resulting from plastic hinging are used. The combination specified must be used for the strong direction of the pier.

1.14 Combination of Horizontal Seismic Forces and Vertical Seismic Forces

Under the condition when the influence of a vertical earthquake must be considered, because of the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions and the vertical direction, the member elastic seismic forces (including moments, axial force and shear force) resulting from analyses in the three perpendicular directions shall be combined to form three load combination cases as follows:

(1) Load Combination Case 1:

Member seismic forces on each of the principal axes of a member shall be calculated by adding 30 percent of the absolute value of the member elastic seismic forces resulting from the analysis in the transverse direction and vertical direction to 100 percent of the absolute value of the corresponding member elastic seismic forces resulting from the analysis in the longitudinal direction.

(2) Load Combination Case 2:

Member seismic forces on each of the principal axes of a member shall be calculated by adding 30 percent of the absolute value of the member elastic seismic forces resulting from the analysis in the longitudinal direction and vertical direction to 100 percent of the absolute value of the corresponding member elastic seismic forces resulting from the analysis in the transverse direction.

(3) Load Combination Case 3:

Member seismic forces on each of the principal axes of a member shall be calculated by adding 30 percent of the absolute value of the member elastic seismic forces resulting from the analysis in the longitudinal direction and transverse direction to 100 percent of the absolute value of the corresponding member elastic seismic forces resulting from the analysis in the vertical direction.

When the foundation and/or column connection forces are determined from plastic hinging of the column, only the member forces induced from lateral longitudinal plastic moments and lateral transverse plastic moments, respectively, need to be considered. The resulting forces need not be combined as specified in this section.

2 Dynamic Analysis Method

2.1 Scope of Application

Bridges having any of the following conditions must perform the seismic analysis and design not only by static approach but also by dynamic approach.

(1) Multi-span continuous bridges which have distinct vibration characteristics for each individual segment due to abrupt or unusual changes in pier type,

height, stiffness, geometry and soil condition.

(2) Bridges with long periods (more than 1.5sec) or tall piers (more than 40m)

(3) Bridges with a small curvature radius.

(4) Bridges which have abrupt changes in span length or whose distribution of weight is uneven

(5) Bridges constructed in the form of a brand-new type which have not experienced a strong earthquake.

(6) Bridges which are constructed overlying a soil stratum of soft clays and silts.

(7) Other bridges having complex response behavior under an earthquake condition .

2.2 Design Ground Acceleration

For bridges which require the use of dynamic analysis, the design ground horizontal acceleration coefficient Z_d can be calculated according to the following formula:

$$Z_d = \frac{ZI}{1.2\alpha_y F_u} \tag{26}$$

In order to prevent the pier from yielding under the occurrence of an intermediate earthquake, the design ground horizontal acceleration coefficient Z_d given in formula (26) shall not be less than Z_d^*:

$$Z_d^* = \frac{ZI}{3.0\alpha_y} \tag{27}$$

2.3 Modified Acceleration Response Spectrum Factors and Correction Factors

The definition of a modified acceleration response spectrum factor of the dynamic analysis is equivalent to that of the static analysis. When the design ground acceleration is governed by equation (26), $ZI/1.2\alpha_y$ shall be used as the correction factors for the dynamic analysis; whereas when equation (27) governs, $ZIF_u/3.0\alpha_y$ shall be used.

Due to the differences in damping ratios corresponding to the equivalent stiffness of the superstructure, substructure and foundation, respectively, the complex modal damping ratios can be calculated by any recognized method. For all those soil profile types with a damping ratio other than 5%, including Taipei Basin, the dimensionless acceleration response spectrum coefficient can be calculated by multiplying the value listed in Table 1 by a damping correction factor C_D as stated in equation (28).

$$C_D = \frac{1.5}{40\xi + 1} + 0.5 \tag{28}$$

where ξ is the damping ratio. It should be noted that formula (28) may be applied only to those bridge structures located on soil types I or II and with a period greater than 0.15sec or to those located on soil type III or the Taipei Basin and with a period greater than 0.2sec. On the other hand, those with a period less than 0.03, C_D should be taken as 1.0. For any other structures with a period other than the ones specified above, the values of C_D should be obtained by linear interpolation.

2.4 Bridge Dynamic Analysis Model

When performing a dynamic analysis, the bridge structure should be modeled realistically to ensure that the simulations of geometrical shape, weight distribution, member section properties and soil-structure interaction effect are accurate. The bridge should be modeled as a three-dimensional space frame in which each joint and node has six degrees of freedom, three translational and three rotational. For curved bridges, the longitudinal motion shall be directed along a chord connecting the abutments, and the transverse motion shall be applied normalized to the chord.

2.5 Multimode Spectral Analysis

For bridges with irregular geometry, the multimode spectral analysis should be adopted to perform the dynamic analysis in order to account for the coupling effects between the longitudinal and transverse seismic responses and the effect of higher modes.

The response should at least include the effects of a number of modes equivalent to three times the number of spans, and the participating effective mass along the longitudinal and transverse direction respectively should exceed 90% of the total mass of the bridge. However, if the number of modes considered already exceeds 25 modes and the shortest period considered is already smaller than 0.25 sec, then the requirements specified above need not be satisfied.

The maximum member forces and displacements should be estimated by combining the respective response quantities from the individual modes by a recognized method. The combination method must be able to account for the coupling effects between individual modes.

2.6 Combination of Two Perpendicular Horizontal Seismic Forces

Due to the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions, the combination effect of member elastic seismic forces resulting from analyses in the two perpendicular directions must be considered. The combination rule for dynamic analysis is the same as that for static analysis.

If the vertical earthquake has an obvious influence on the seismic response of a bridge, the effect of the vertical seismic forces must be considered. The member elastic seismic forces are resulted from the combination of the vertical and the two horizontal seismic forces. The vertical ground acceleration can be taken as 2/3 of the horizontal ground acceleration, and the dimensionless acceleration response spectrum coefficient can be calculated from Table 3.

Appendix: Additional notes on the Structural Design Method for Buildings and Bridges in Taiwan

a. Format:

- ■ Working Stress Design: Allowable Stress ≥ Actual Stress
- ■ Ultimate Strength Design: Ultimate Member Strength ≥ Required Member Strength
- ■ Limit State Design: Ultimate Lateral Strength ≥ Required Lateral Strength
- ■ Other Design Method

b. Material Strength (Concrete and Steel)

Specific Compressive Strength of Concrete: 21-35 MPa

Specific Yield Strength of Rebar: 280-420 MPa

Ultimate Strength of Structural Steel: 400-570 MPa

c. Strength Reduction Factor

(1) Flexure without Axial Load --- 0.9

(2) Axial Tension and Axial Tension with Flexure ---- 0.9

(3) Axial Compression and Axial Compression with Flexure

--- 0.7-0.75 for Reinforced Concrete

--- 0.65-0.7 for Steel and Steel Reinforced Concrete

(4) Shear and Torsion

--- 0.85 for Reinforced Concrete

--- 0.75 for Steel and Steel Reinforced Concrete

(5) Bearing on Concrete --- 0.7

d. Load Factors for Gravity Loadings and Load Combinations

<u>Allowable Stress S in Working Stress Design</u>

(1) S = D (Dead) + L (Live)

(2) S = D + L + E (Earthquake Effect)

(3) S = D + L + W (Wind effect)

Allowable stress is increased 33% for load combinations (2) and (3).

<u>Required Concrete Member Strength U in Ultimate Strength Design</u>

(1) U = 1.4D+1.7L

(2) U= 0.75(1.4D+1.7L+1.7W)

(3) U= 0.9D+1.3W

(4) U= 0.75(1.4D+1.7L+1.87W)

(5) U=0.9D+1.43E

(6) U= 1.4D+1.7L+1.7H (Soil or Water Pressure)

(7) U=0.75(1.4D+1.7L+1.4T) (Temperature, Creep, Shrinkage and Differential Settlement)

(8) U=1.4(D+T)

<u>Required Steel Member Ultimate Strength Y in Limit State Design</u>

(1) Y=1.4D

(2) Y=1.2D+1.6L

(3) Y=1.2D+0.5L+1.6L

(4) Y=1.2D+0.5L+E

(5) Y=0.9D-E

(6) Y=0.9D-1.6W

<u>Required Steel Reinforced Concrete Member Ultimate Strength Y in Limit State Design</u>

(1) Y=1.4D

(2) Y=1.2D+1.6L

(3) Y=1.2D+(0.5L or 0.8W)

(4) Y=1.2D+0.5L+1.3W

(5) Y=1.2D+0.5L+1.0E

(6) $Y=0.9D\pm(1.0E \text{ or } 1.3W)$

e. Typical Live Load Values

Residential Buildings: 2.0 kN/m^2

Office Buildings: 3.0 kN/m^2

Department stores: 5.0 kN/m^2

f. Special Aspects of the Structural Design Method

Structures designed by using Ultimate Strength Design or Limit State Design should perform as follows in three different earthquake intensity levels:

(1) Frequently occurring small earthquake (30-year return period): Structures remain elastic during these earthquakes.

(2) Design base earthquakes (DBE, 475-year return period): Structures become inelastic during these earthquakes. The ductility demands are not larger than the allowable ductility capacity.

(3) Maximum considered earthquake (MCE, 2500-year return period): Structures do not collapse during these earthquakes. The ductility demands are not larger than the ductility capacity.