APPENDIX 3B:

SPECTRAL ACCELERATION MAPS FOR LEVEL 2 EARTHQUAKE GROUND MOTION

(1,000-YEAR RETURN PERIOD)

Appendix 3B presents the spectral acceleration maps for Level 2 Earthquake Ground Motion (1,000-year return period) consisting of:

- Horizontal peak ground acceleration coefficient (PGA)
- Horizontal response spectral acceleration coefficient at 0.20-sec period
- Horizontal response spectral acceleration coefficient at 1.0-sec period

The maps are prepared for the entire Philippine archipelago and the Regional administrative levels.

HORIZONTAL PEAK GROUND ACCELERATION COEFFICIENT (PGA)

(1,000-YEAR RETURN PERIOD)

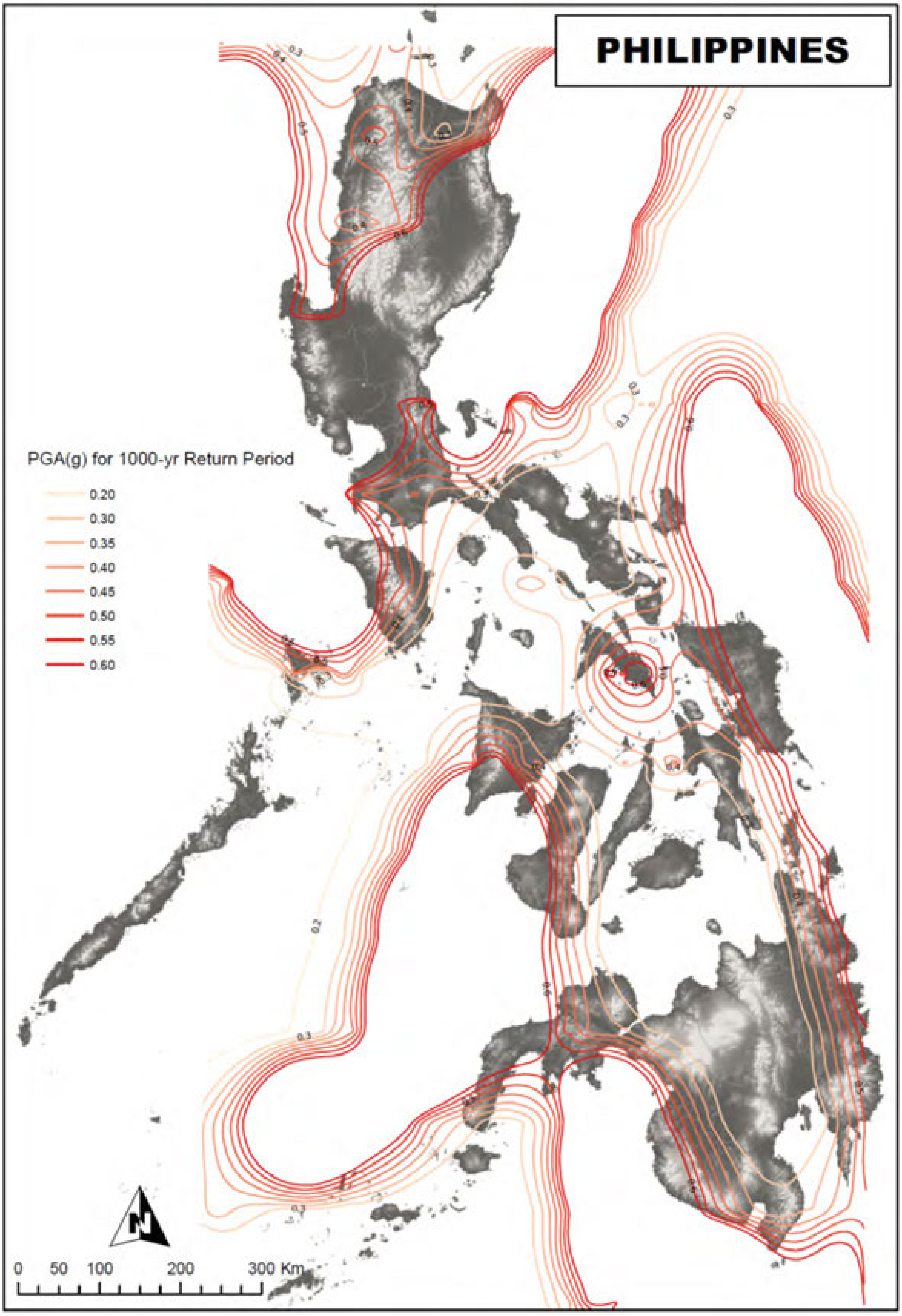


Figure 3B-1 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Entire Country)

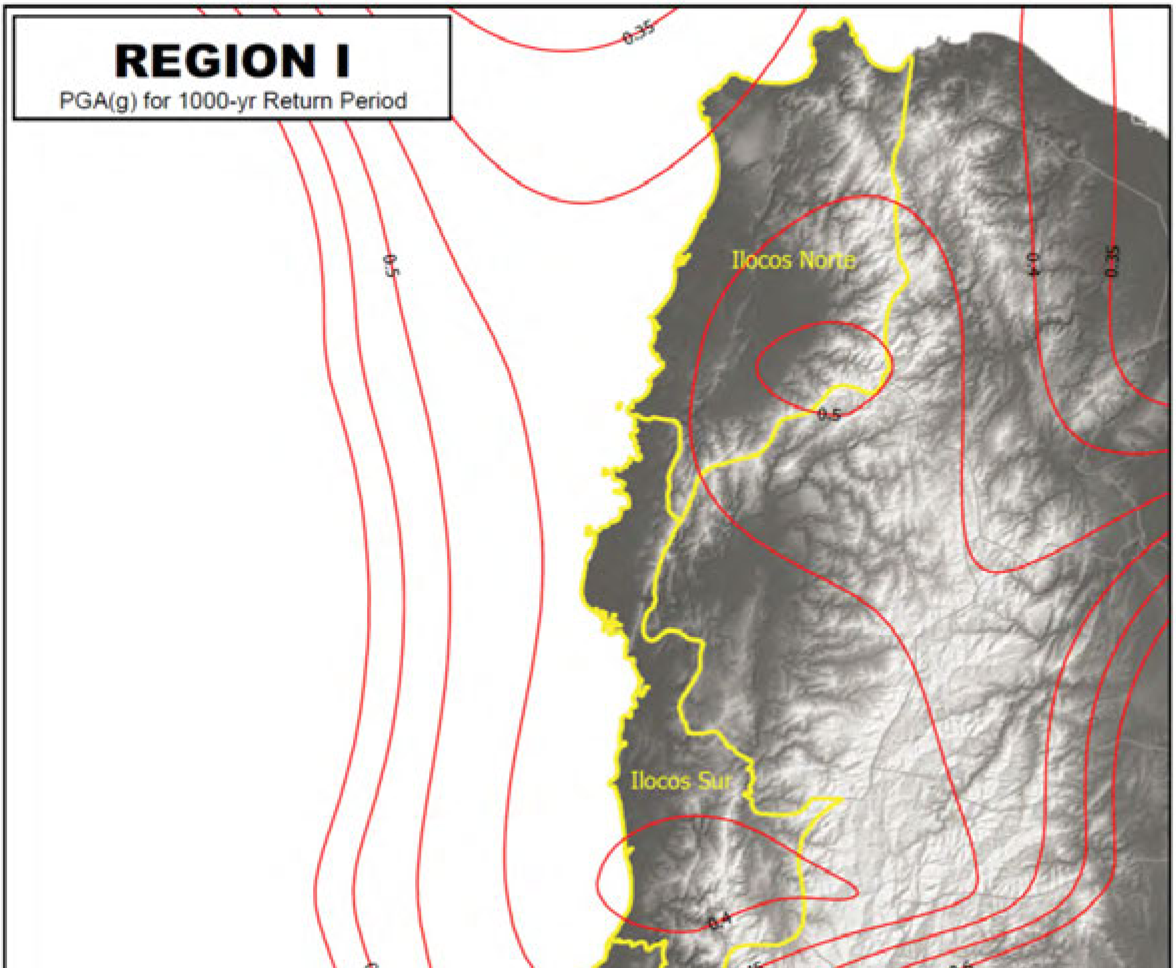


Figure 3B-2 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region I)

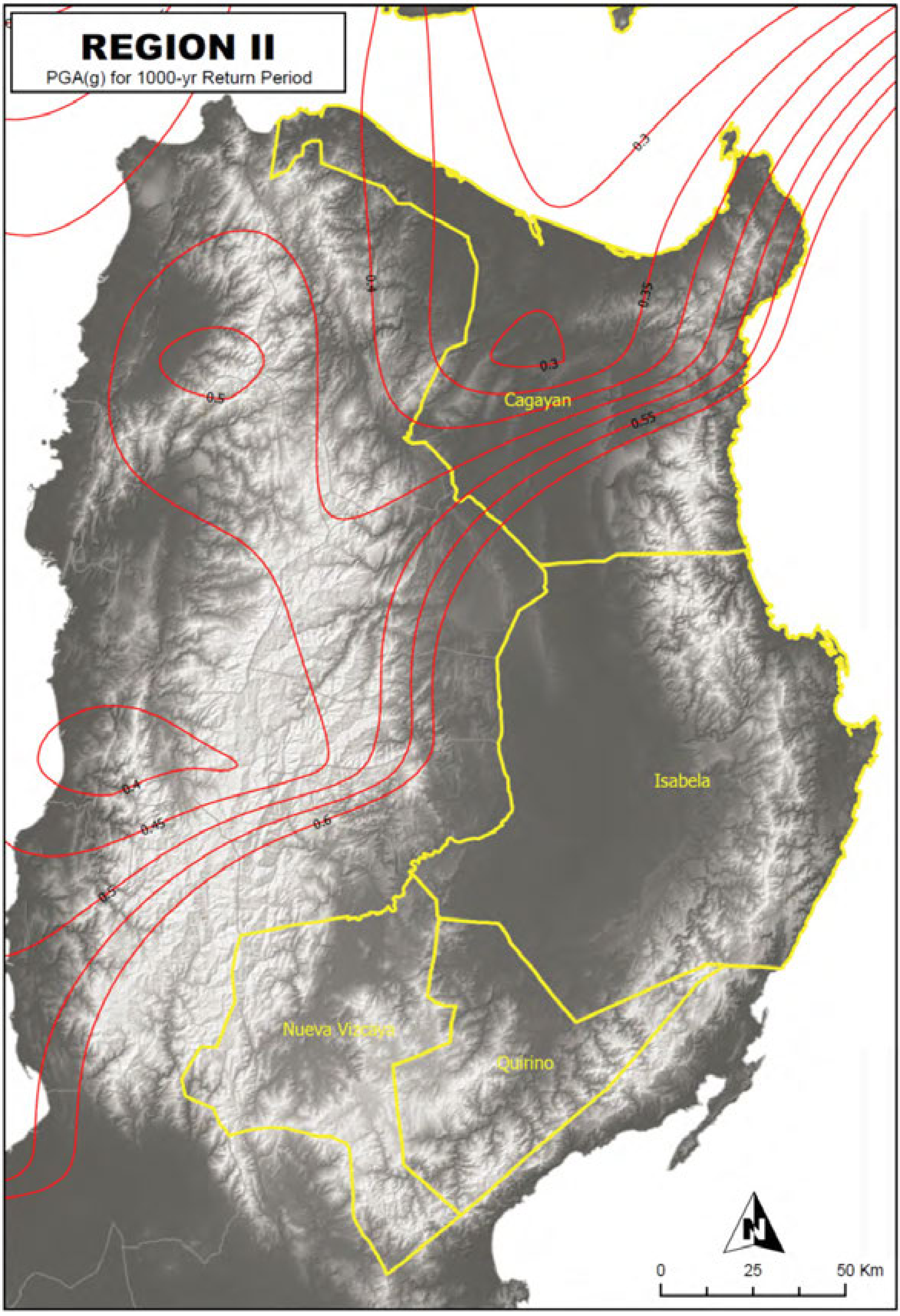


Figure 3B-3 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region II)

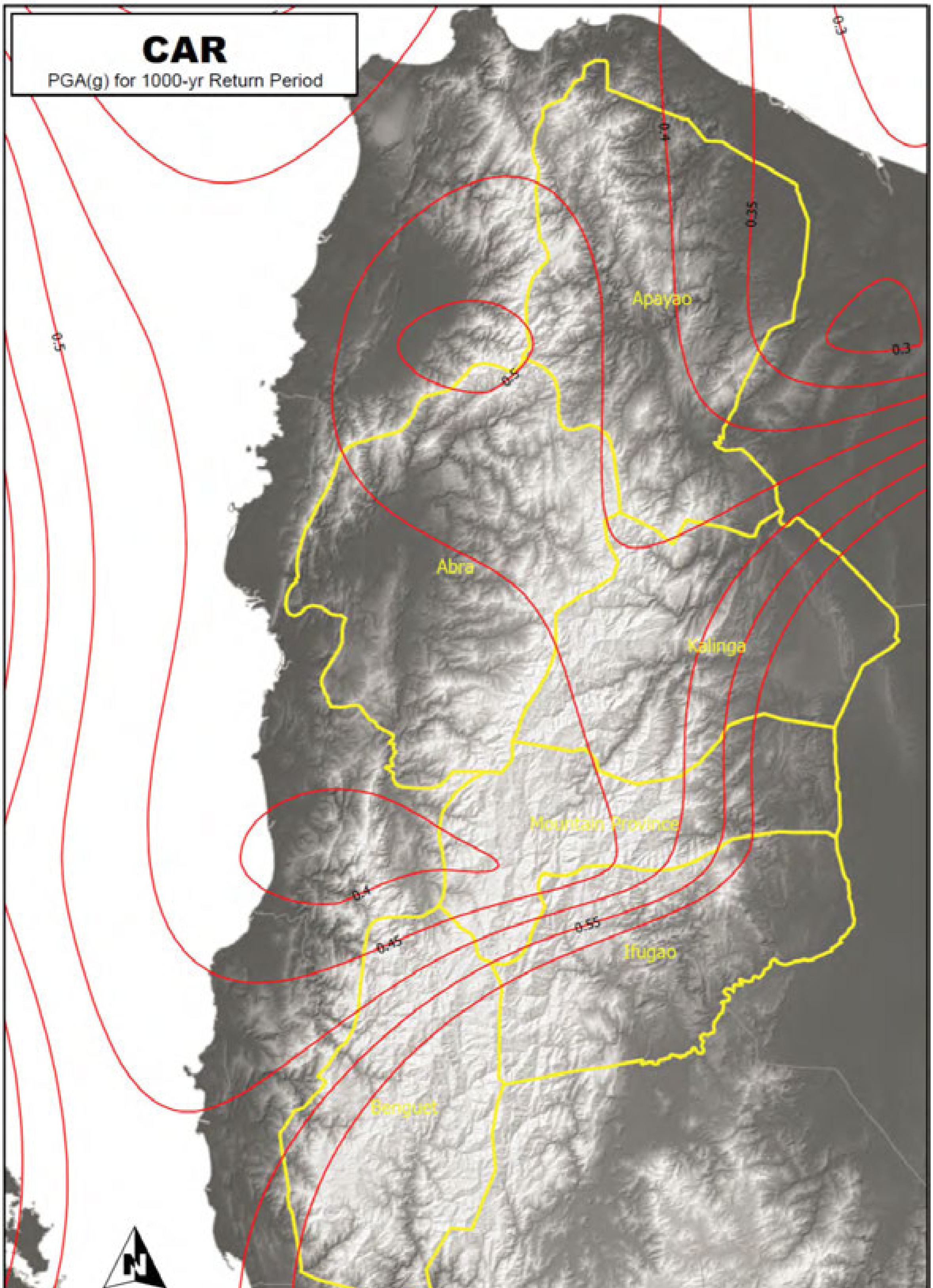


Figure 3B-4 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (CAR)

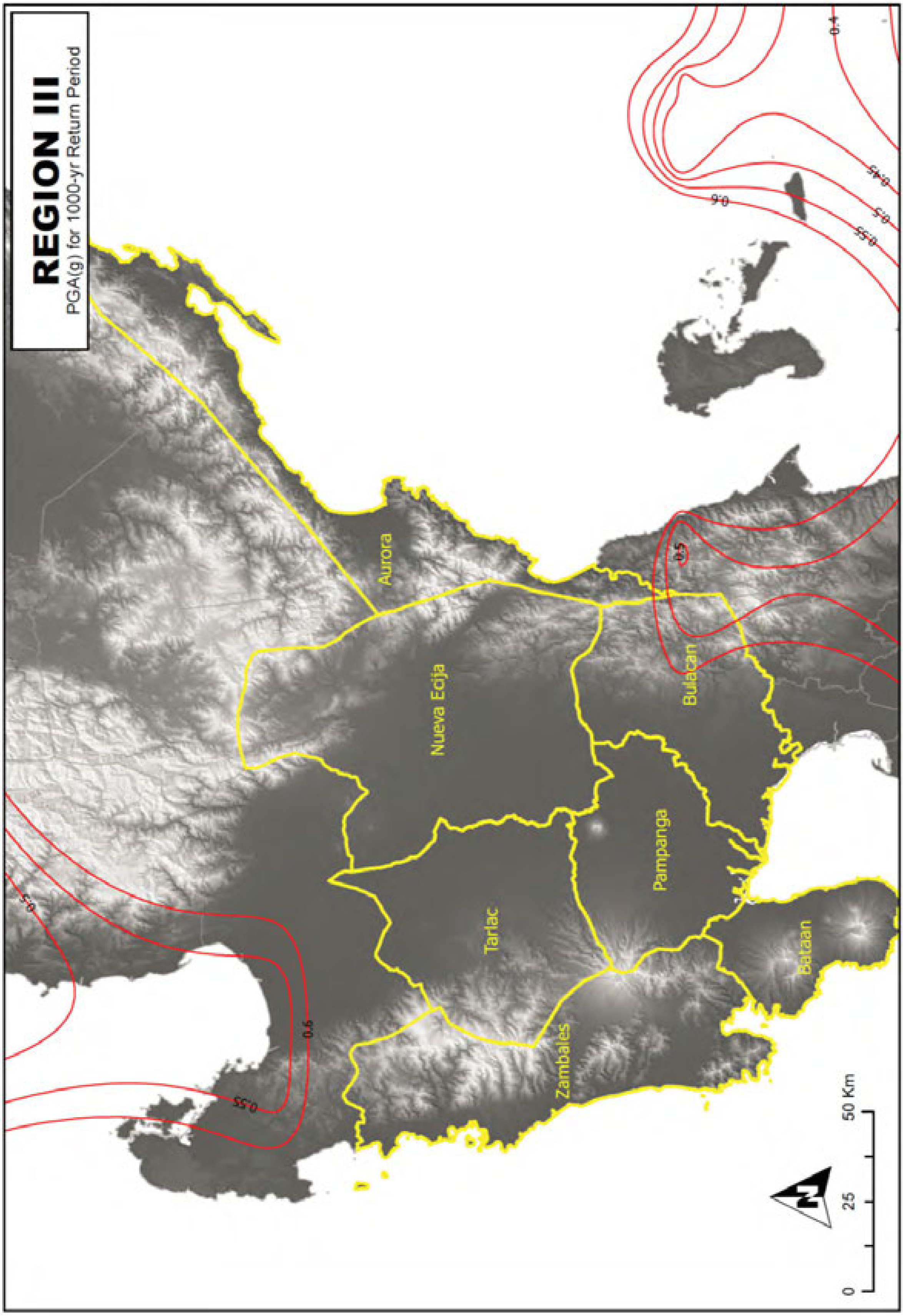


Figure 3B-5 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region III)

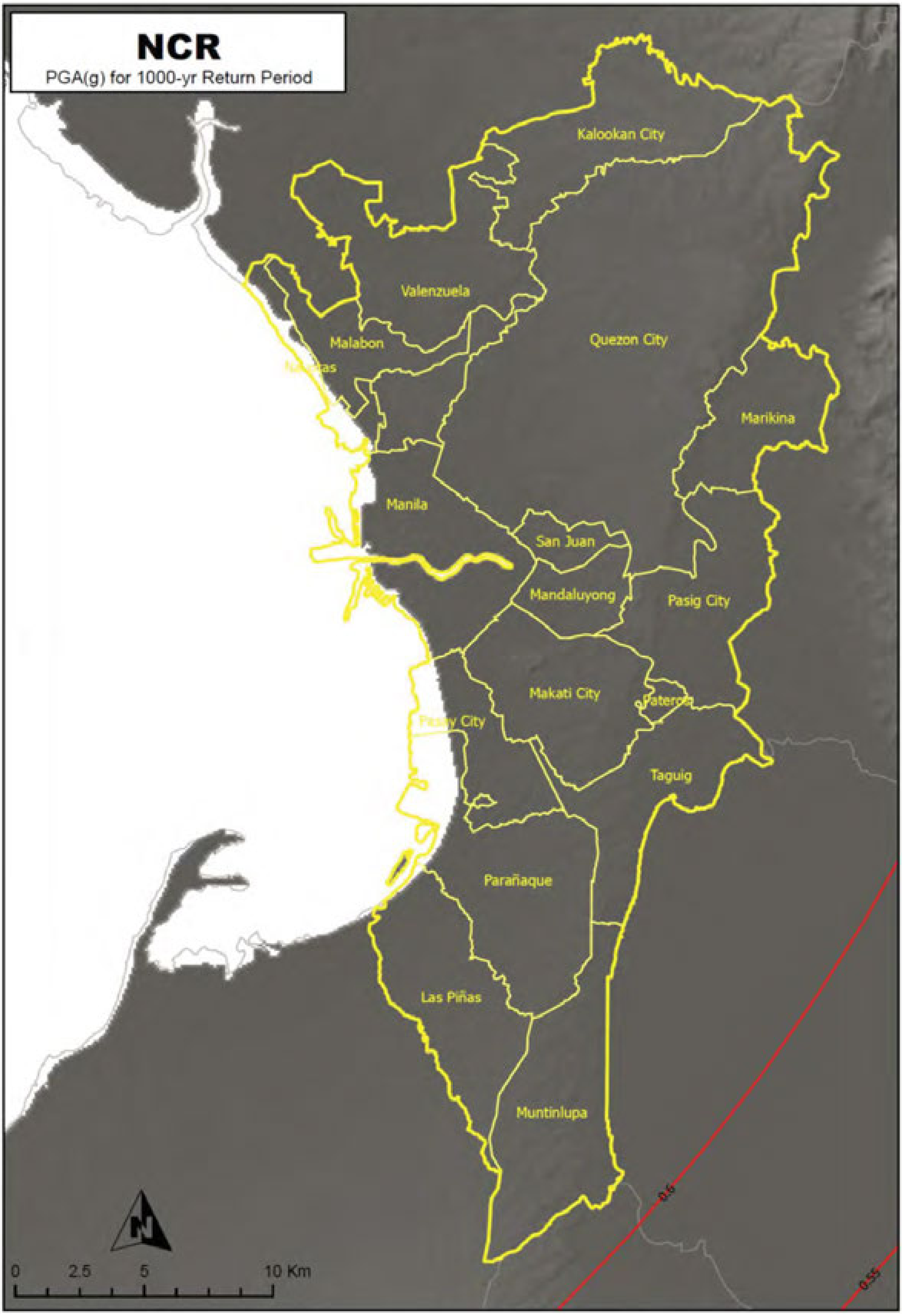


Figure 3B-6 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (NCR)

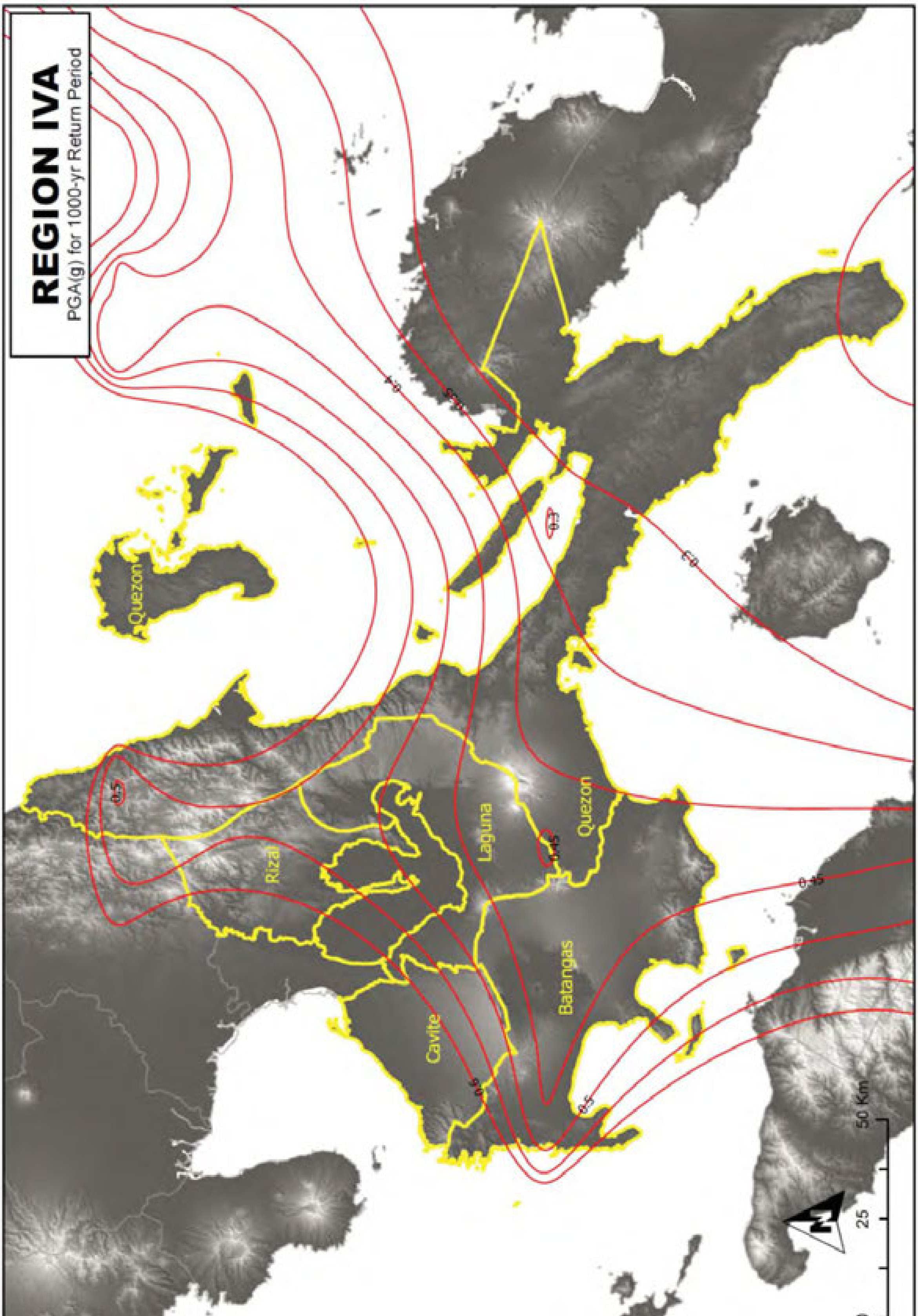


Figure 3B-7 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IV-A)

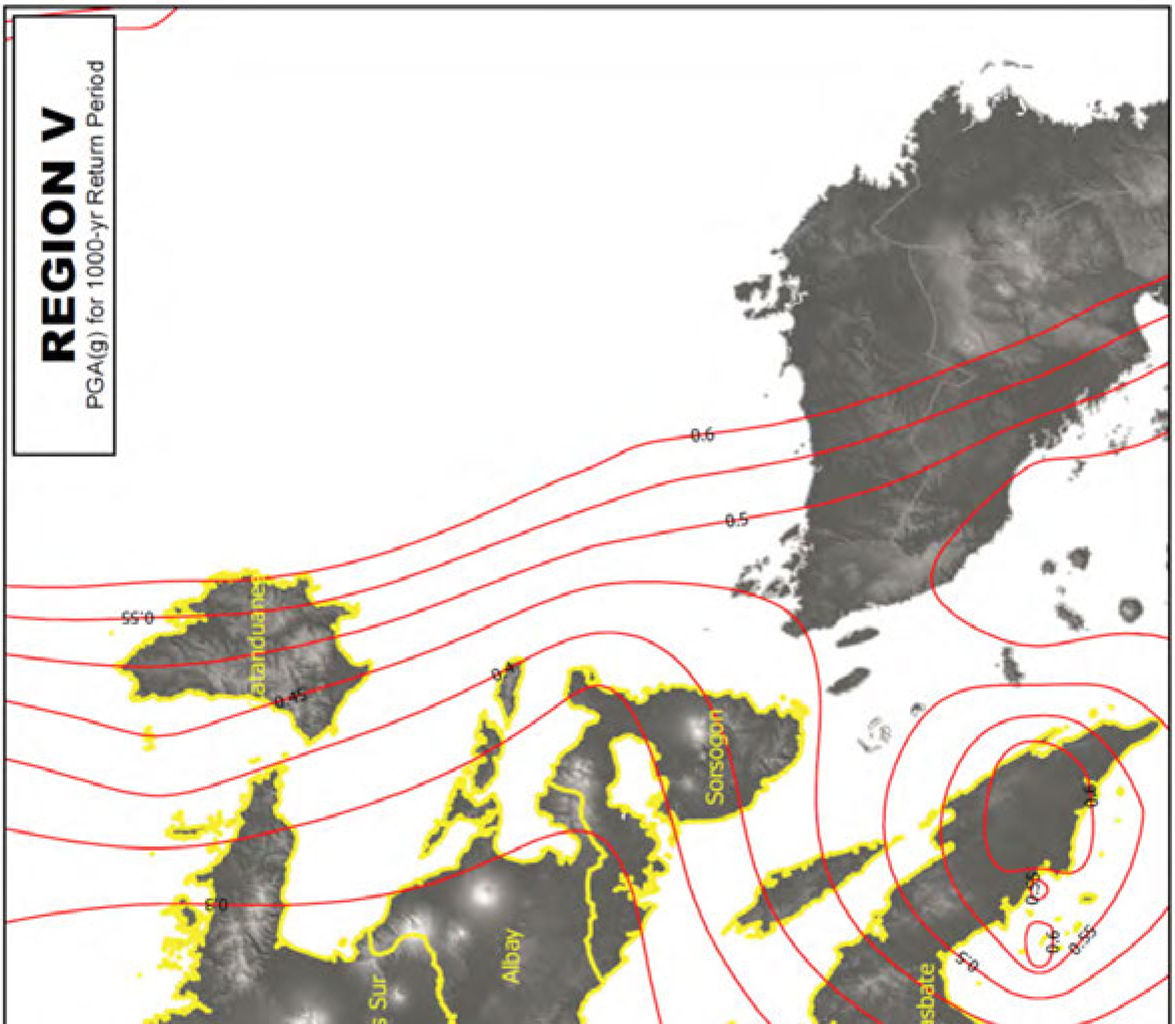


Figure 3B-8 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region V)

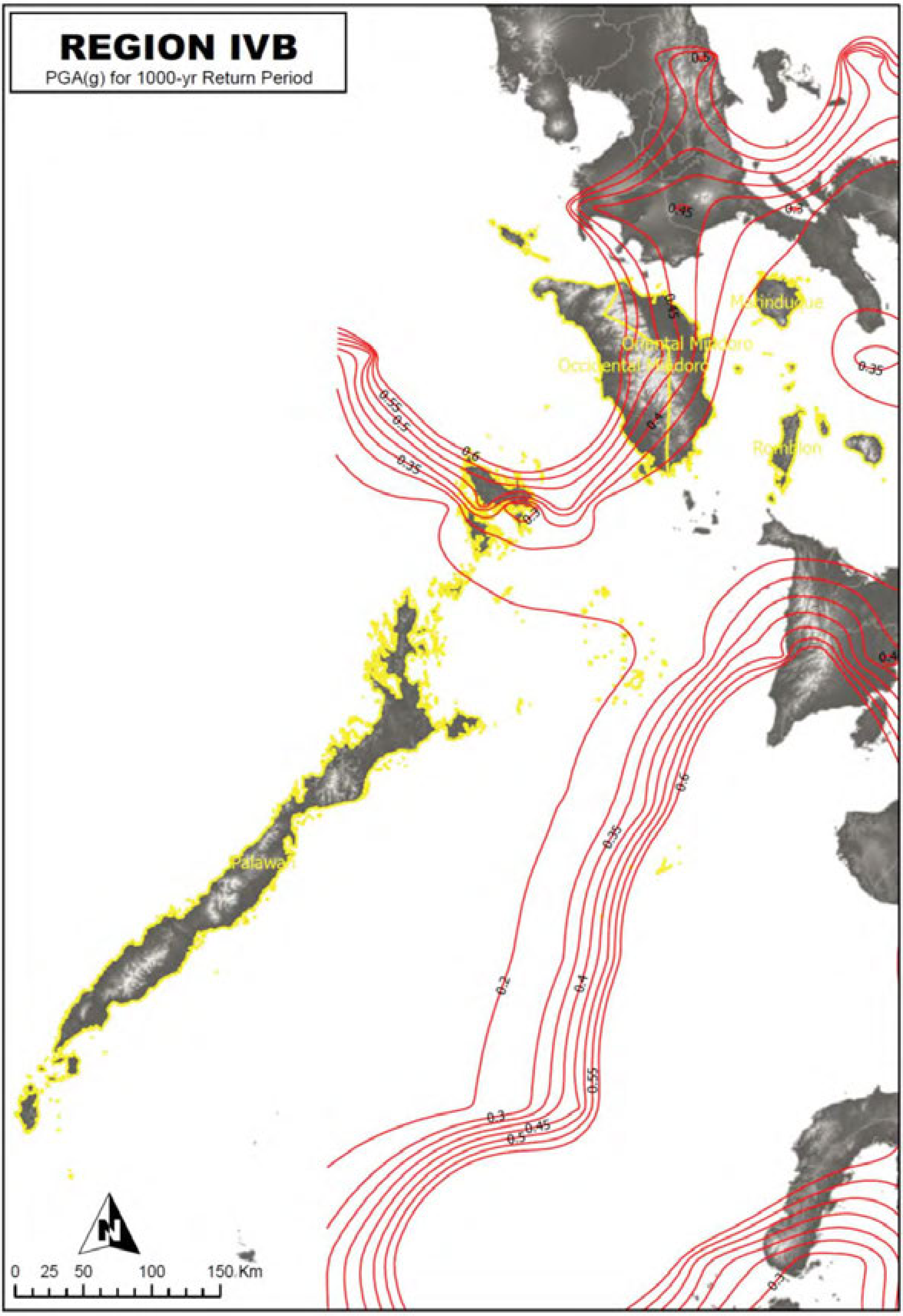


Figure 3B-9 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IV-B)

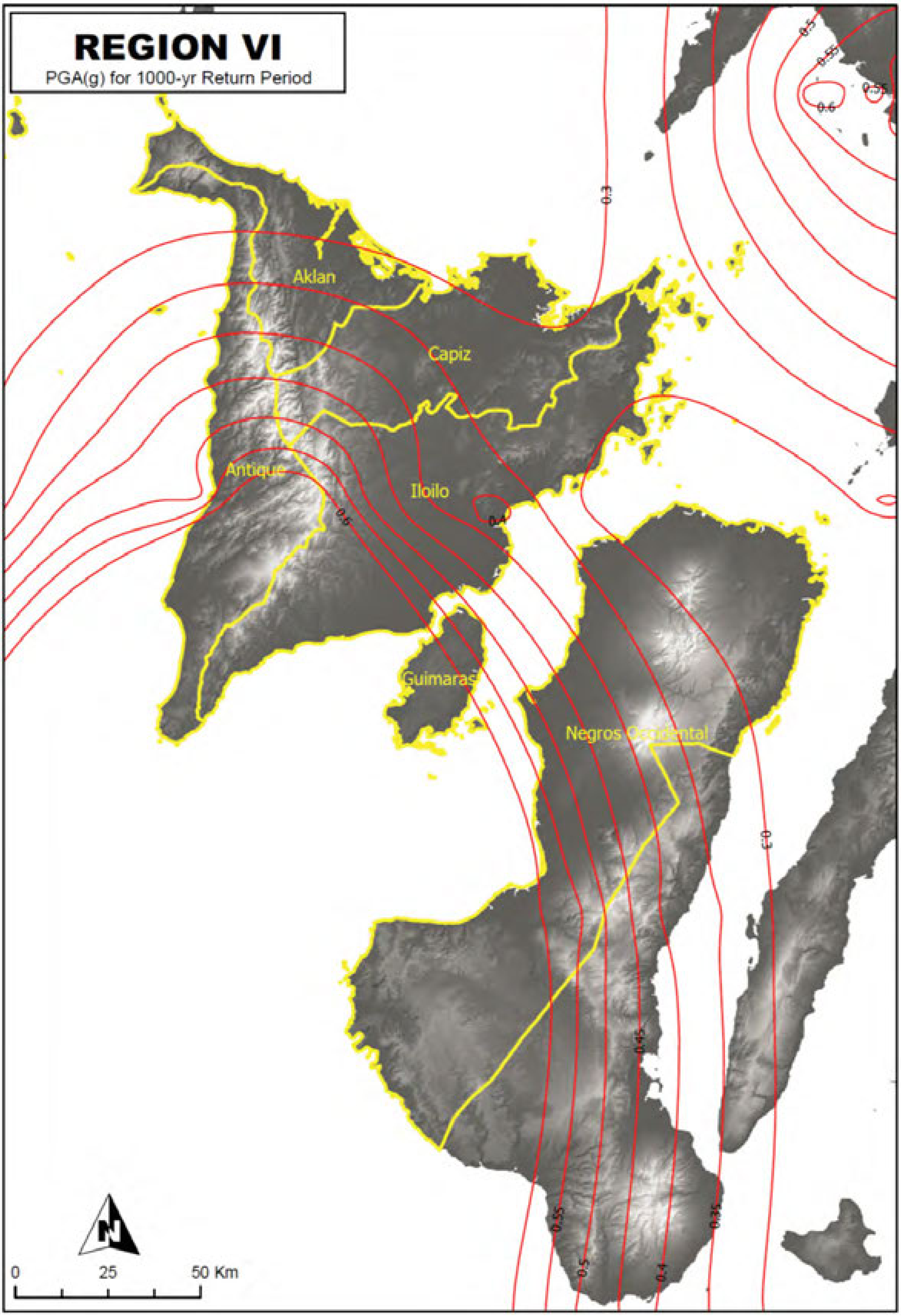


Figure 3B-10 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VI)

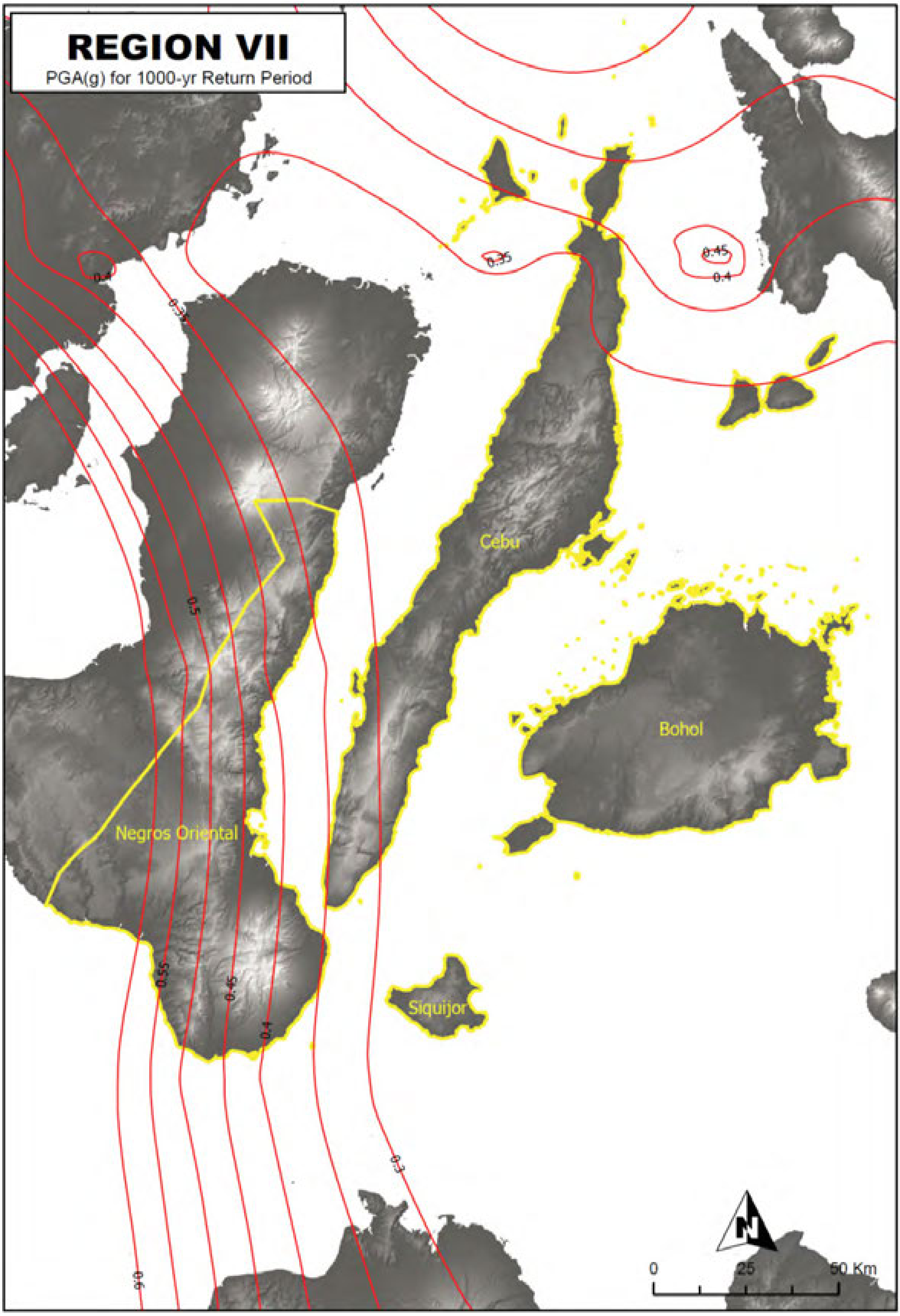


Figure 3B-11 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VII)

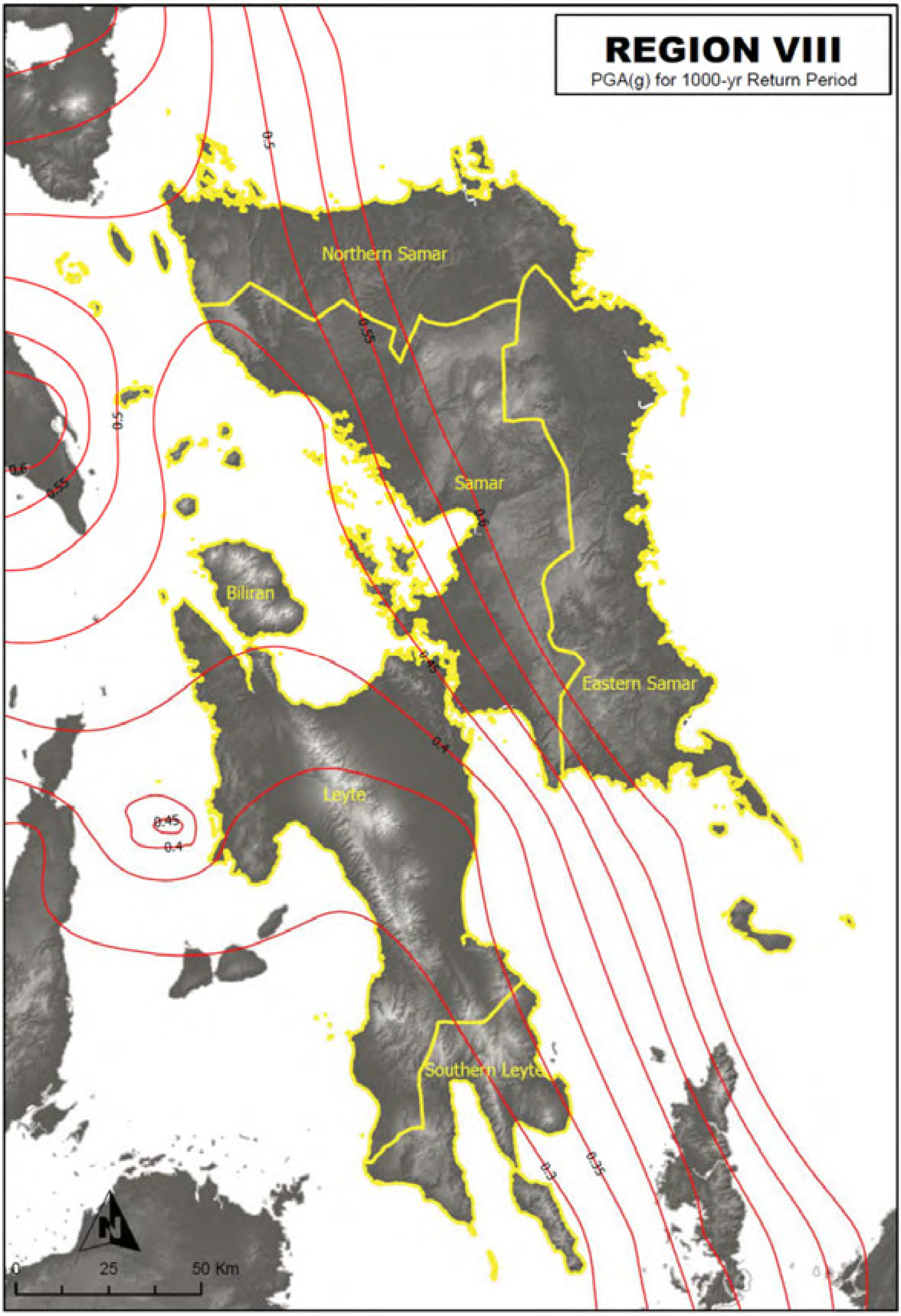


Figure 3B-12 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VIII)

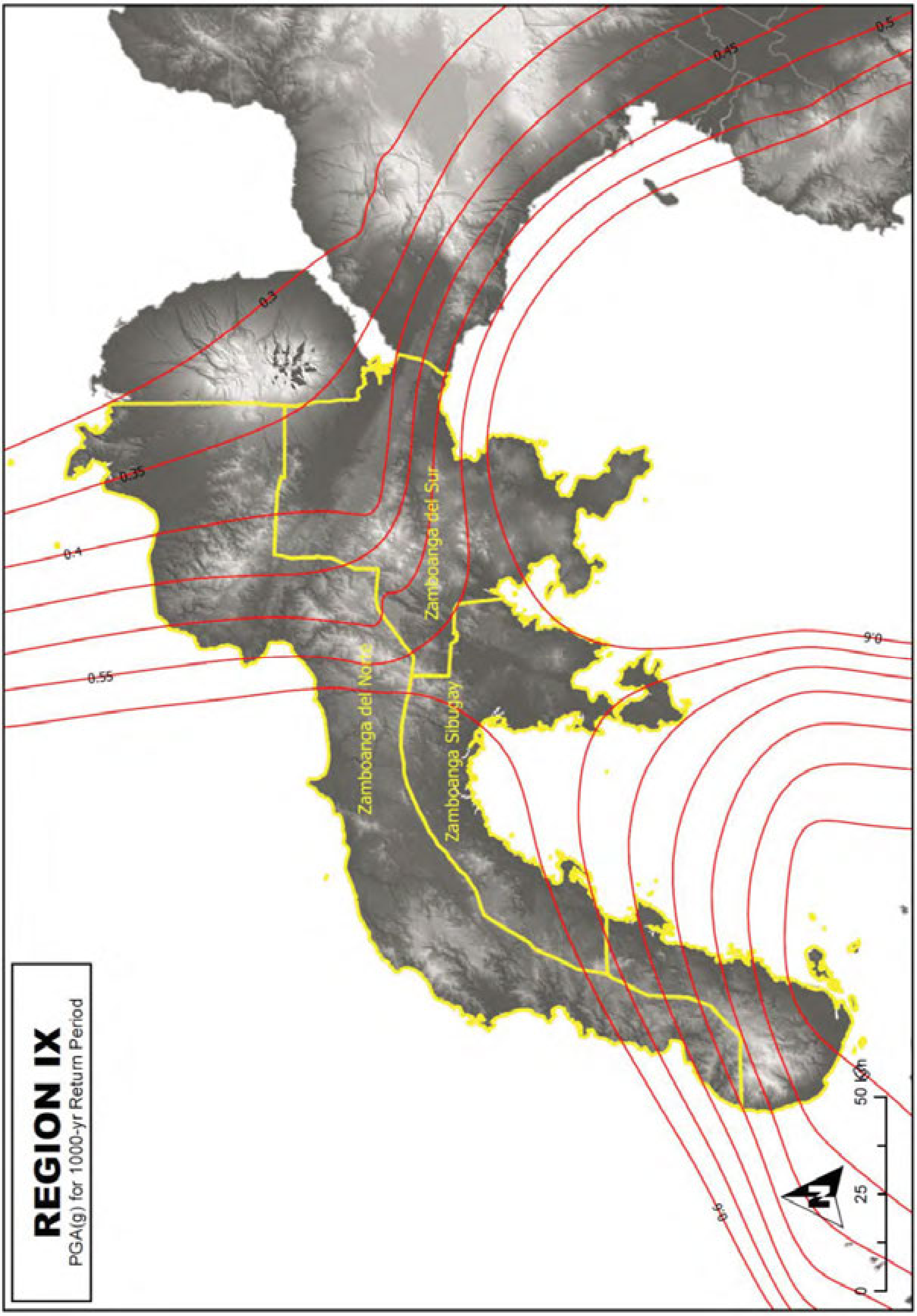


Figure 3B-13 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IX)

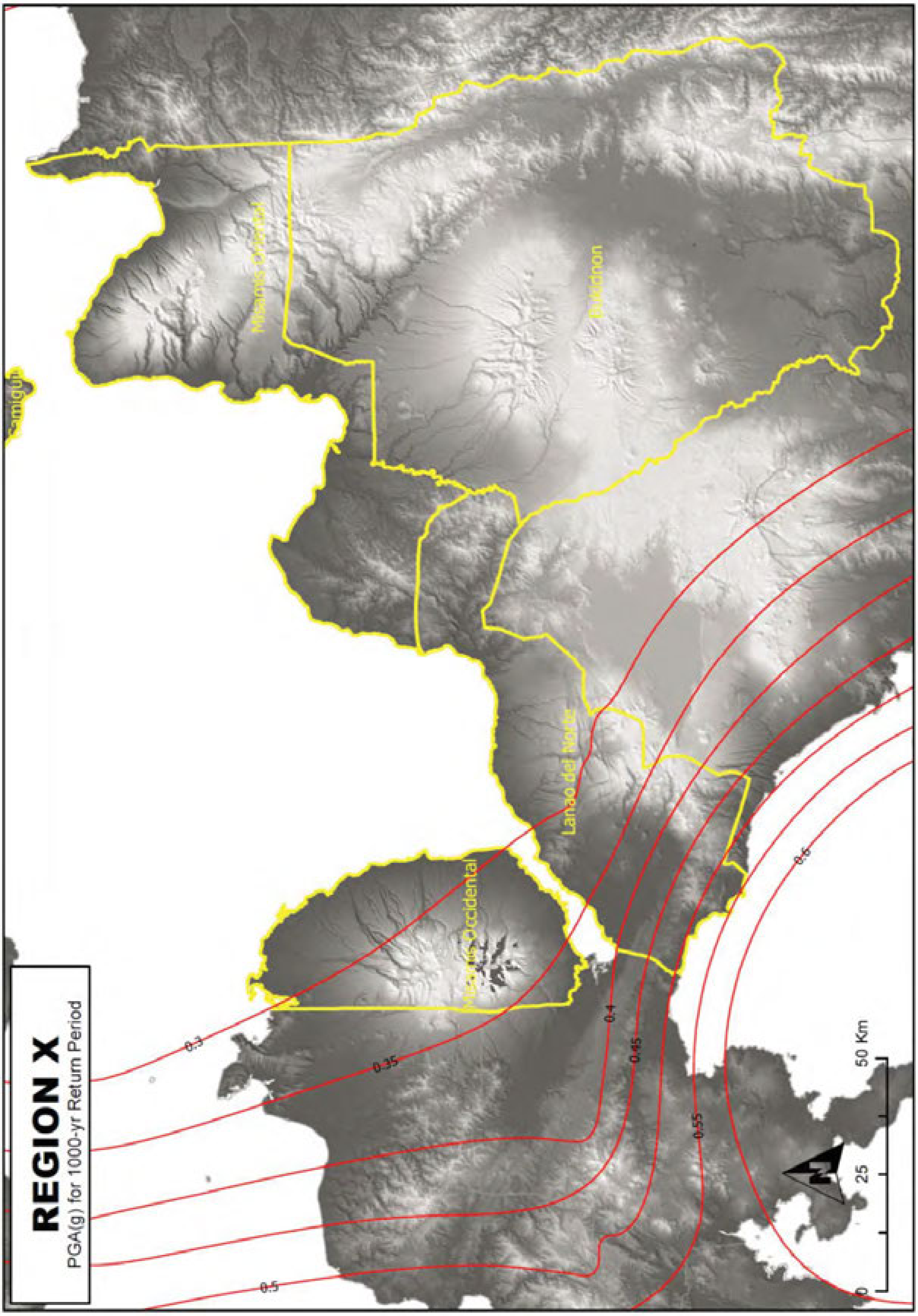


Figure 3B-14 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region X)

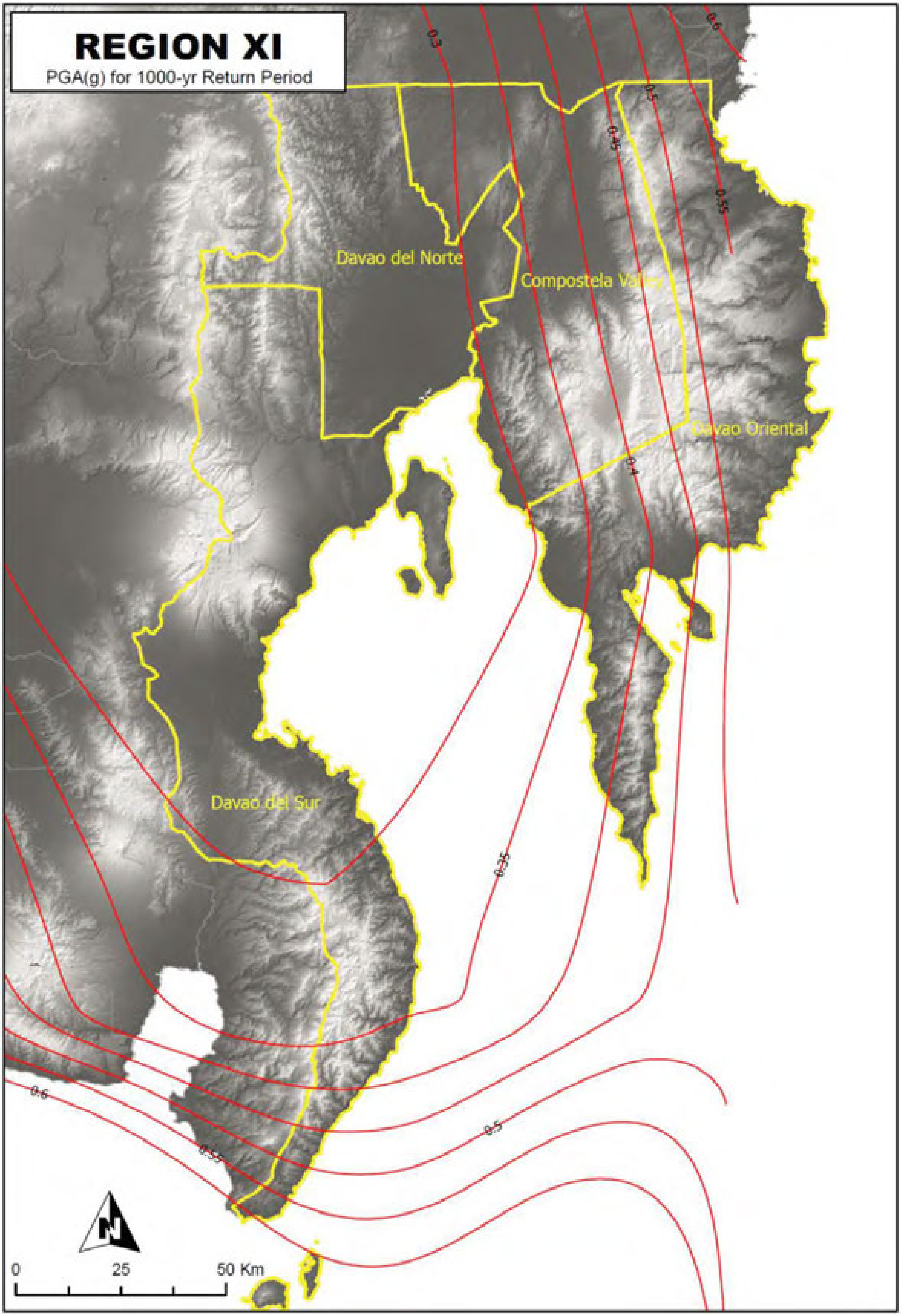


Figure 3B-15 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XI)

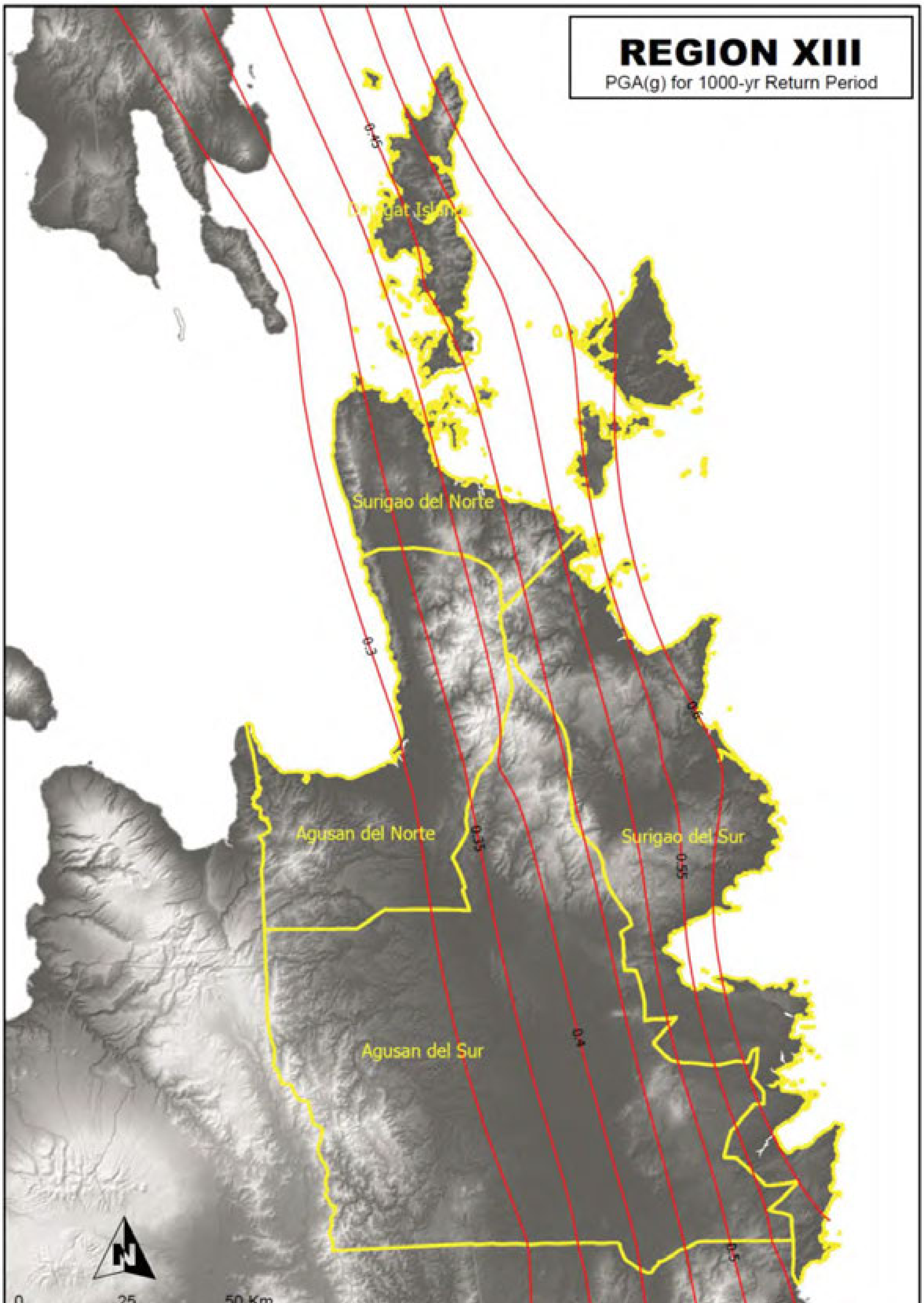


Figure 3B-16 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XIII)

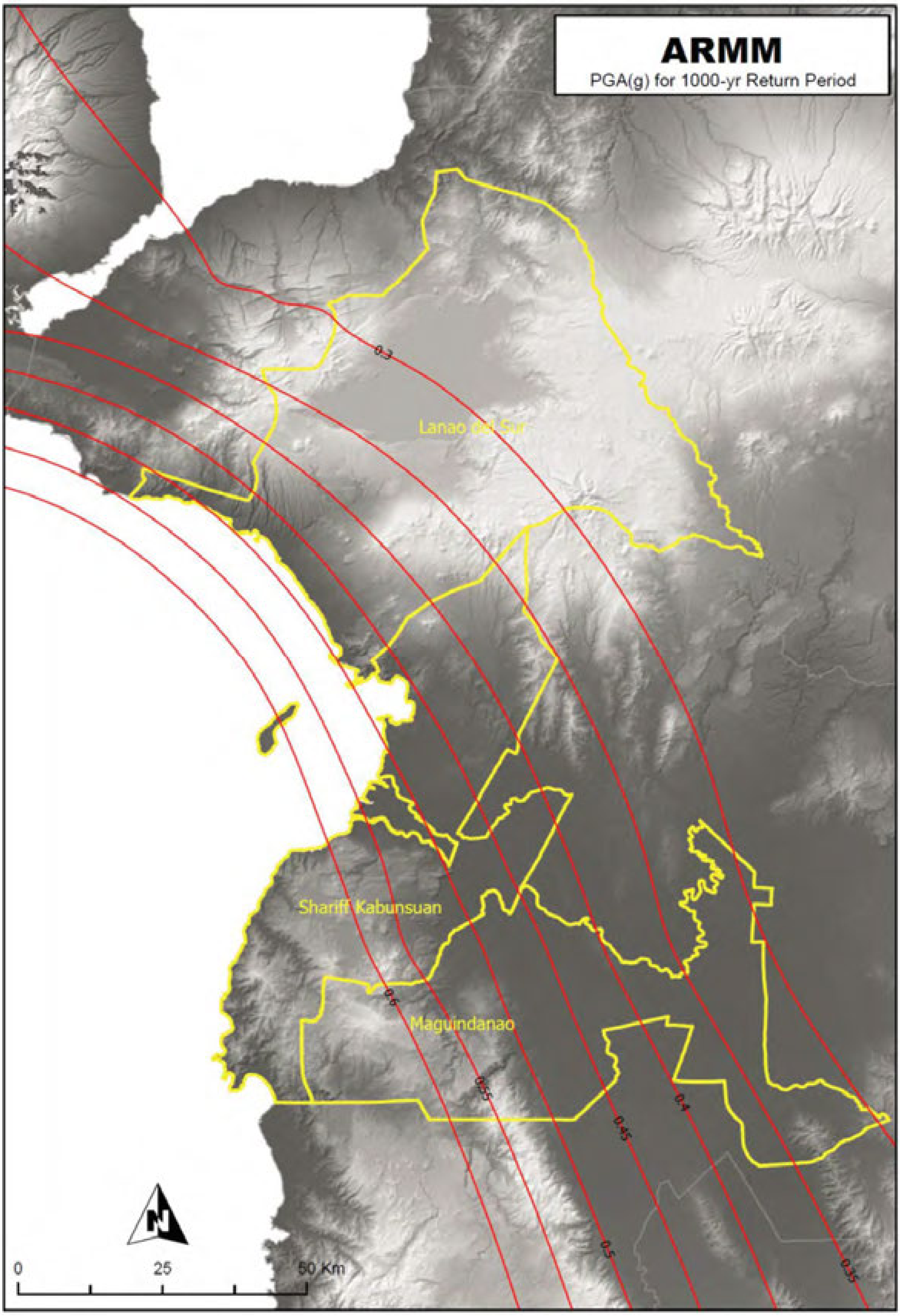


Figure 3B-17 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region ARMM)

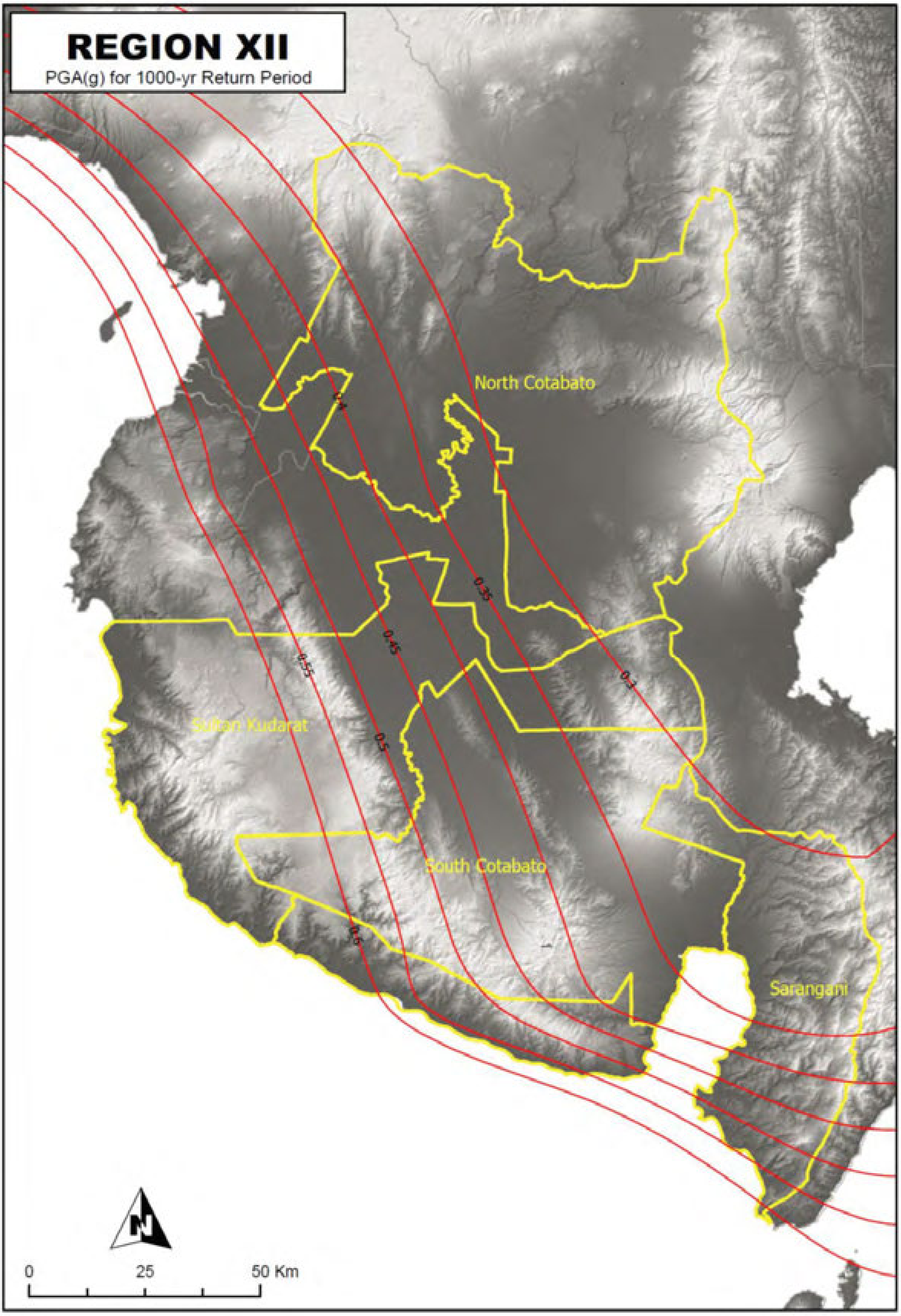


Figure 3B-18 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XII)

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT 0.20-SEC PERIOD

(1,000-YEAR RETURN PERIOD)

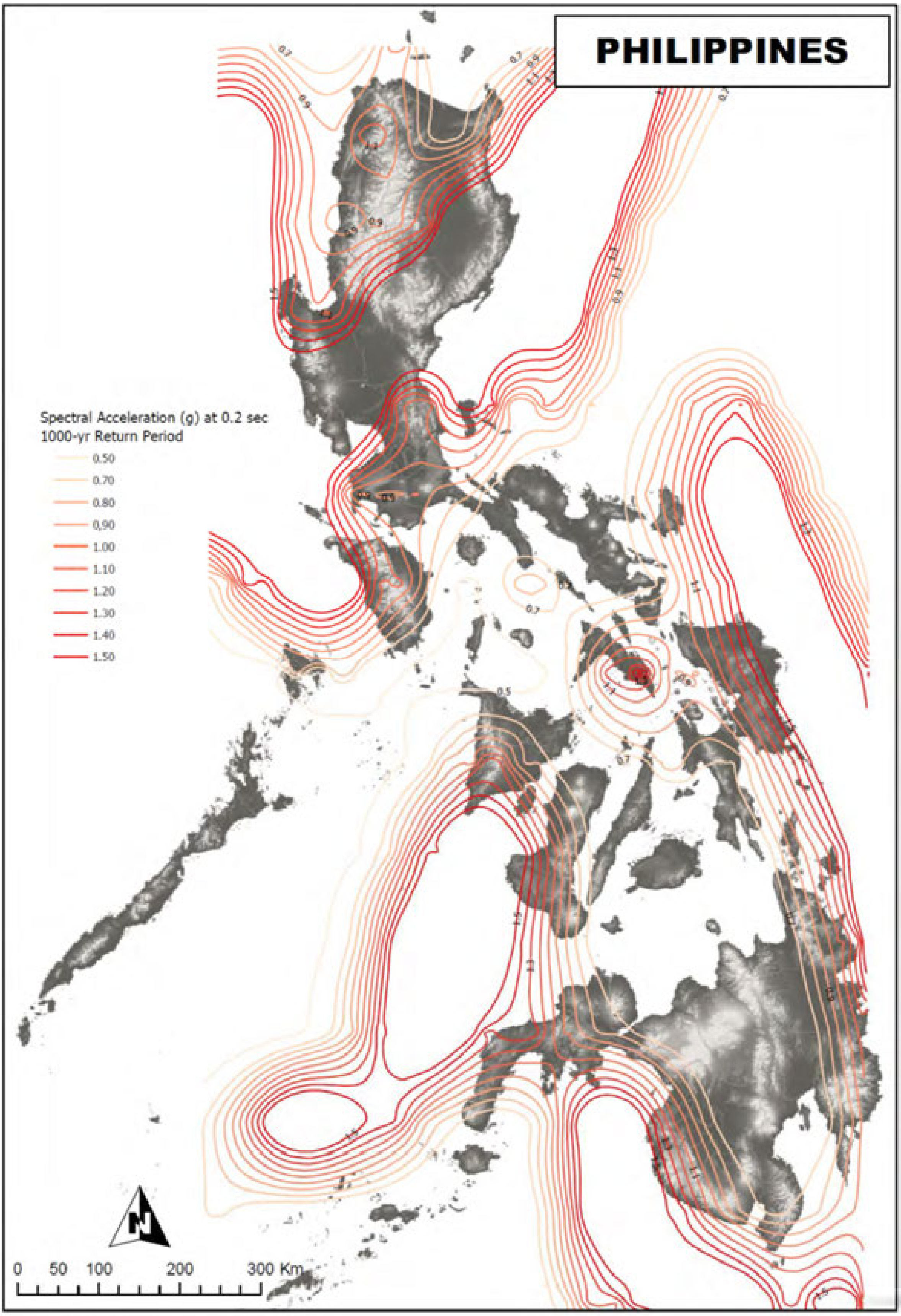


Figure 3B-19 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Entire Country)

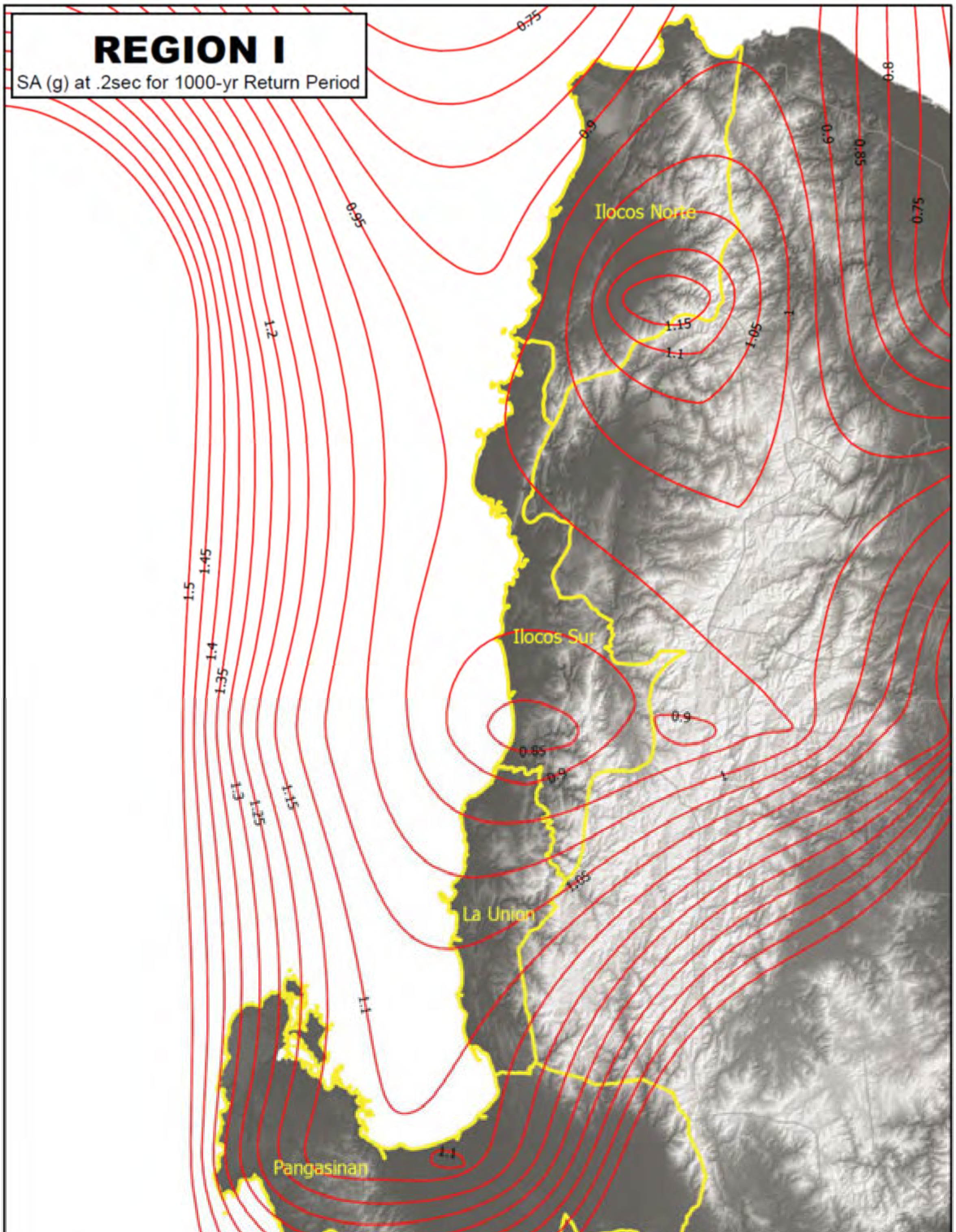


Figure 3B-20 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) Level 1 Earthquake Ground Motion (Region I)

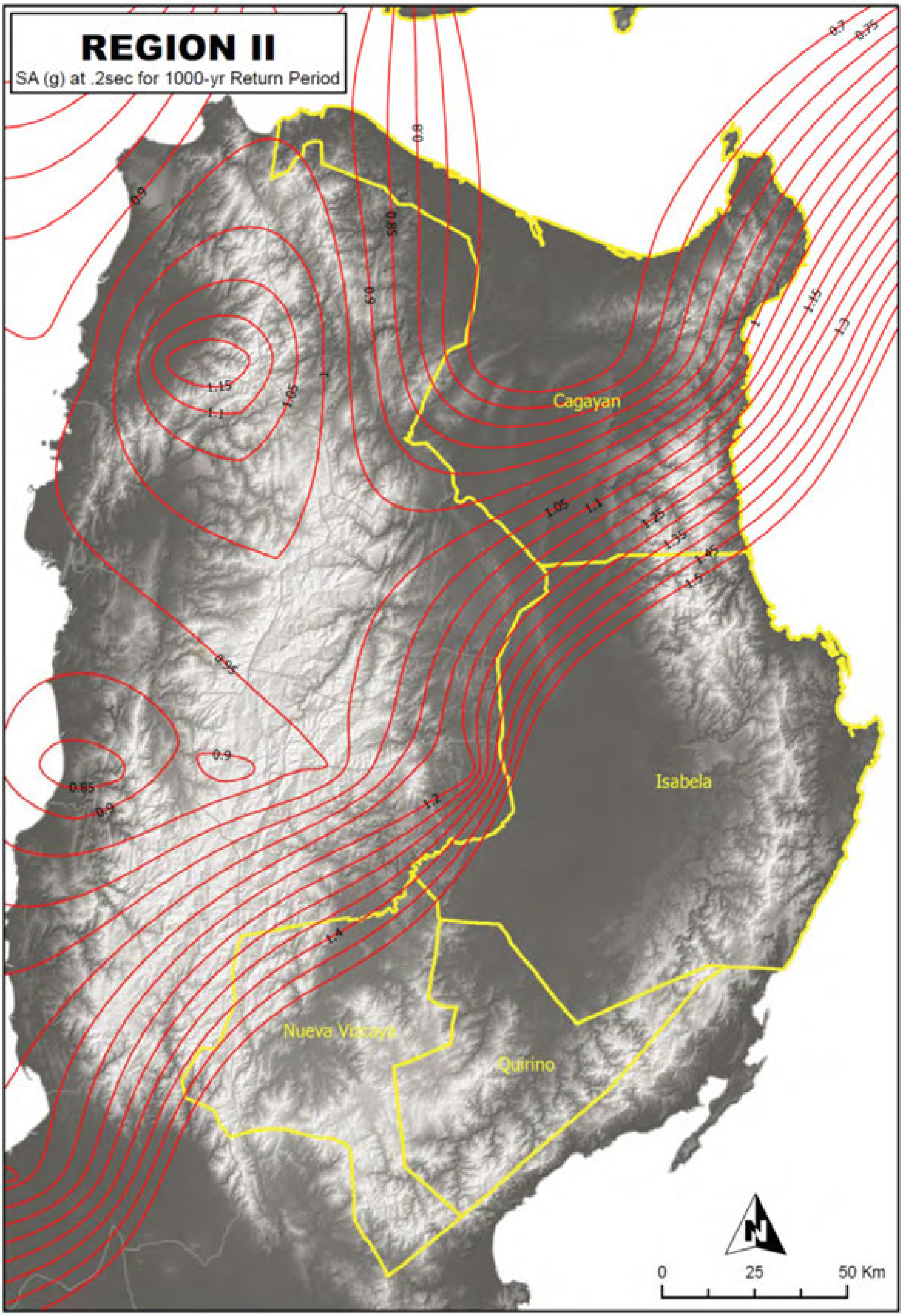


Figure 3B-21 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region II)

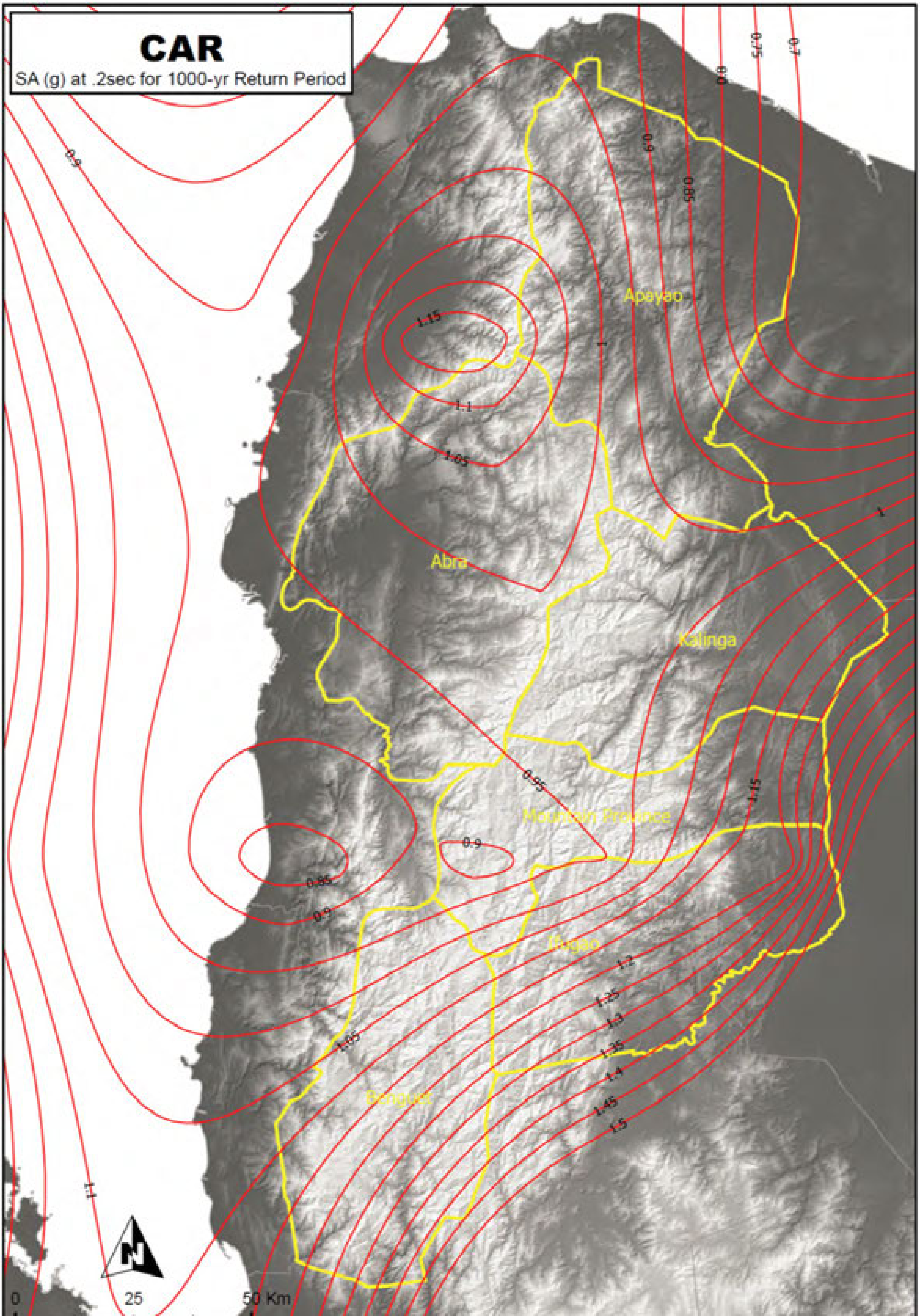


Figure 3B-22 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (CAR)

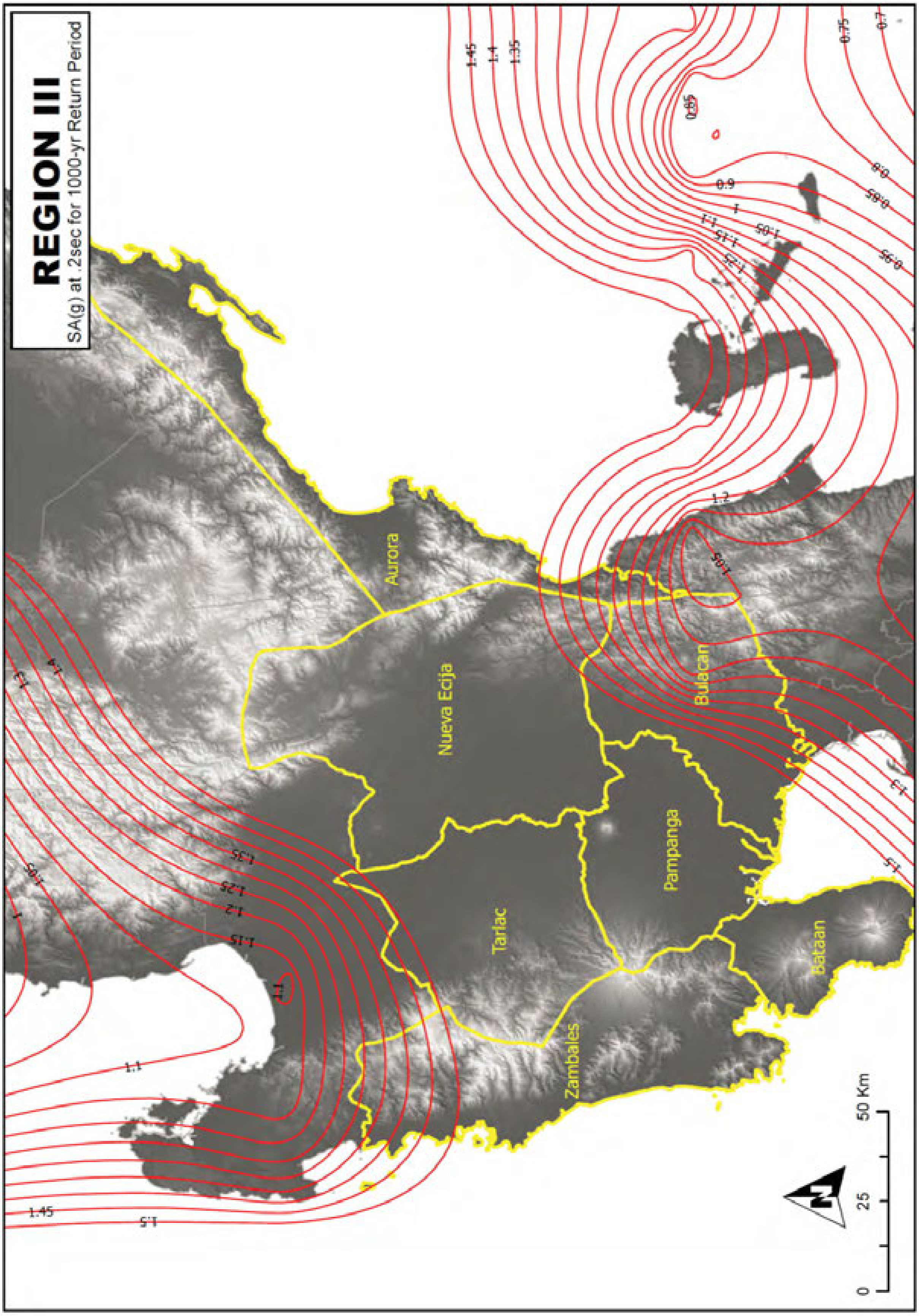


Figure 3B-23 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region III)

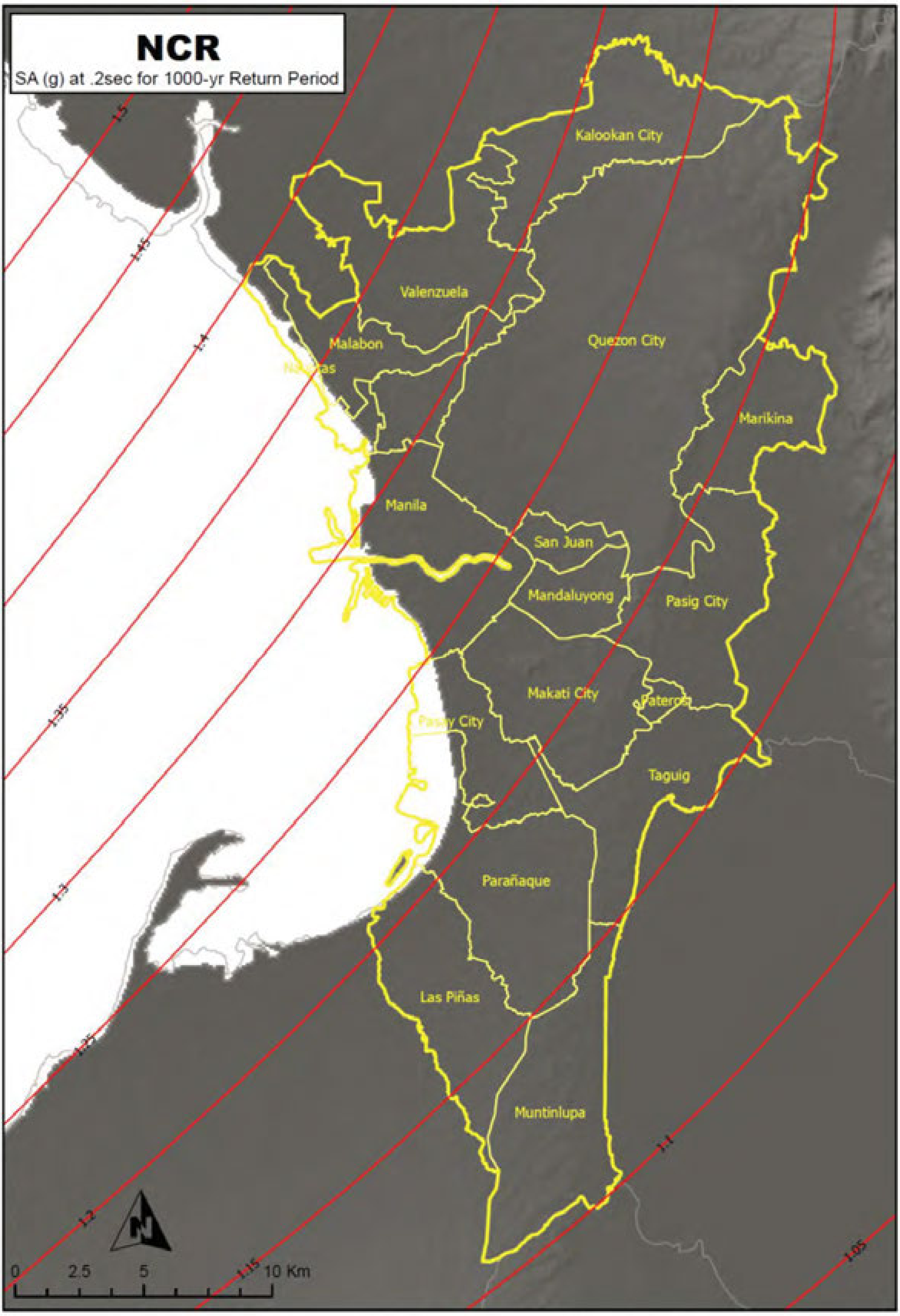


Figure 3B-24 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (NCR)

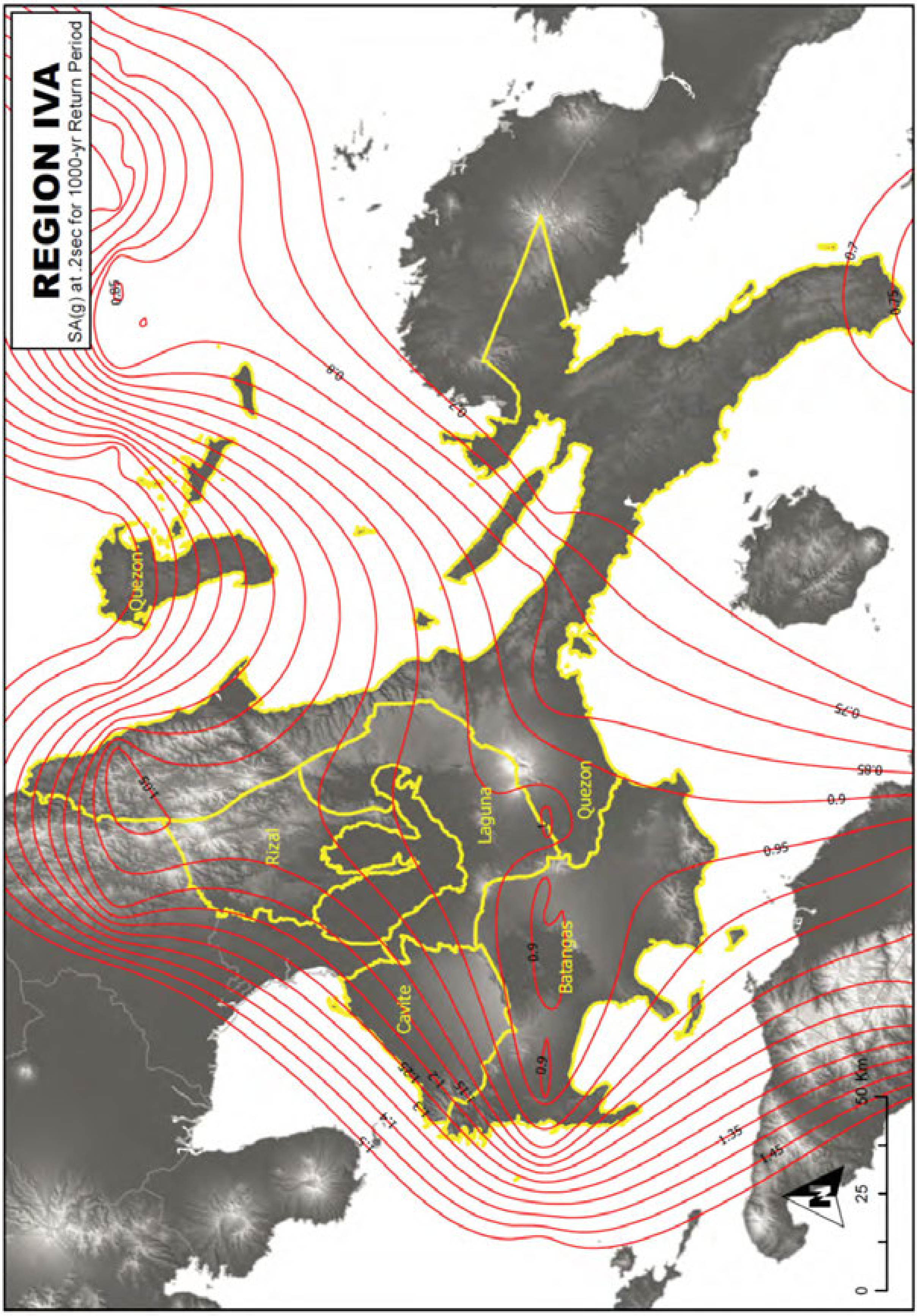


Figure 3B-25 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IV-A)

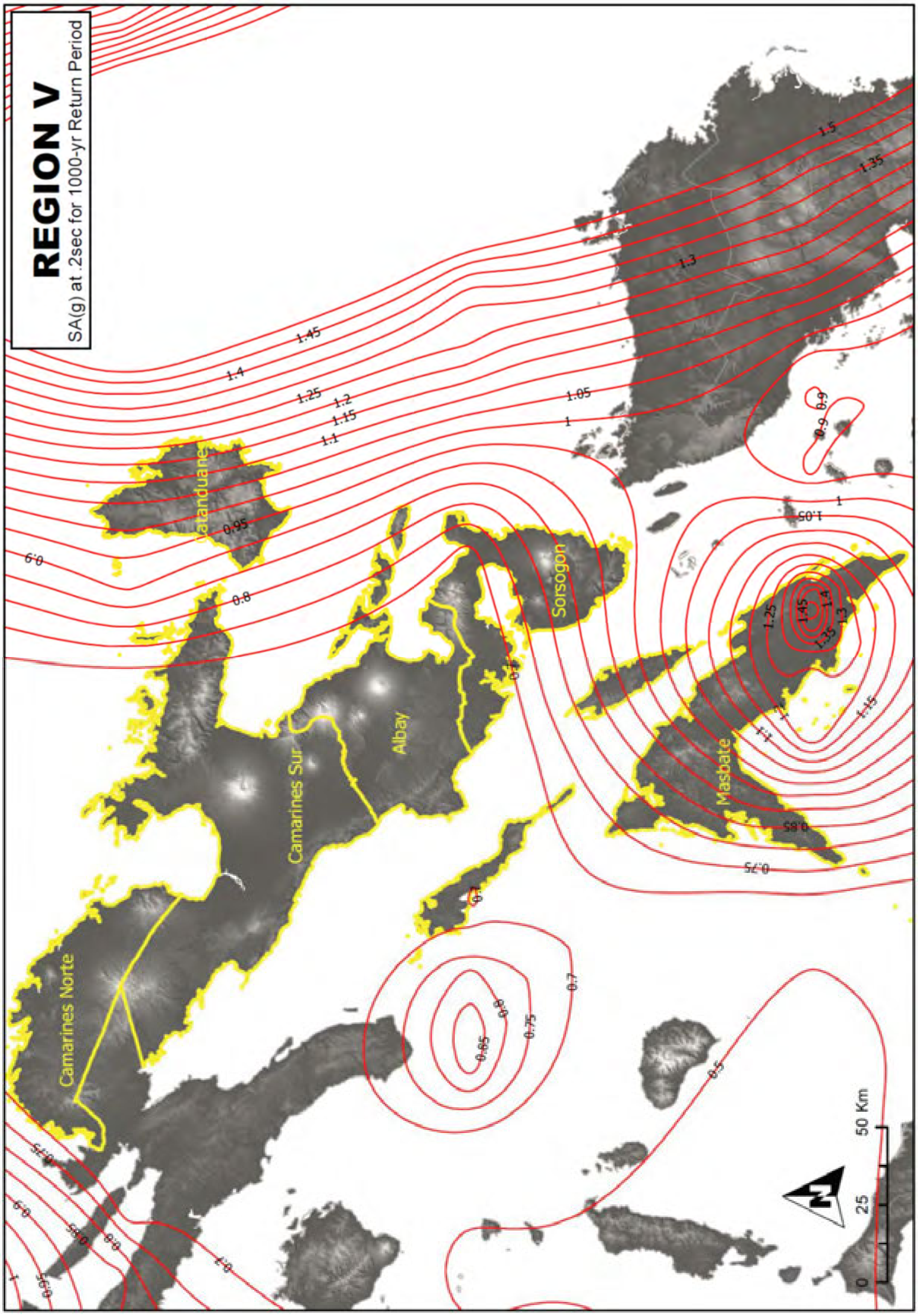


Figure 3B-26 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region V)

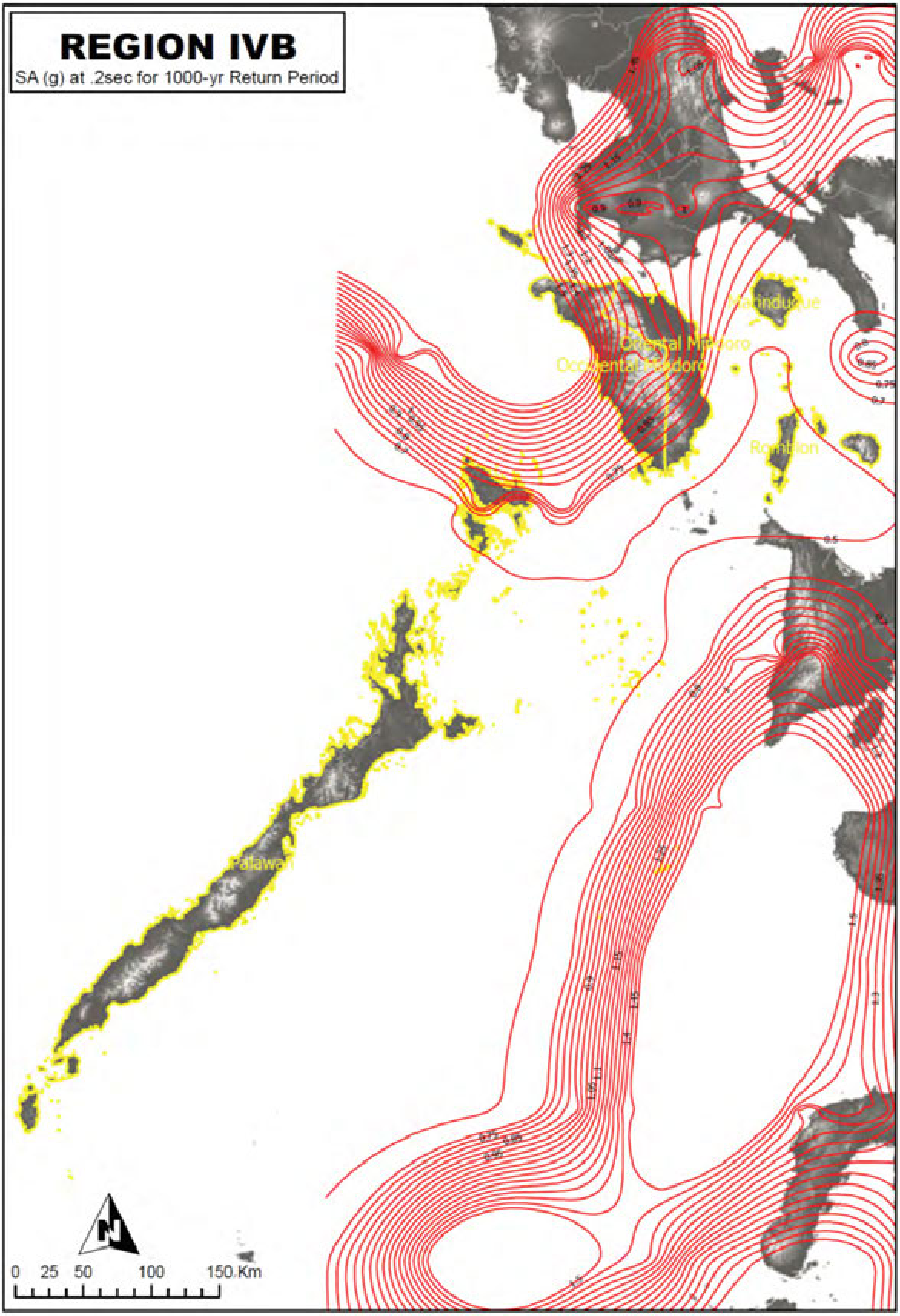


Figure 3B-27 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IV-B)

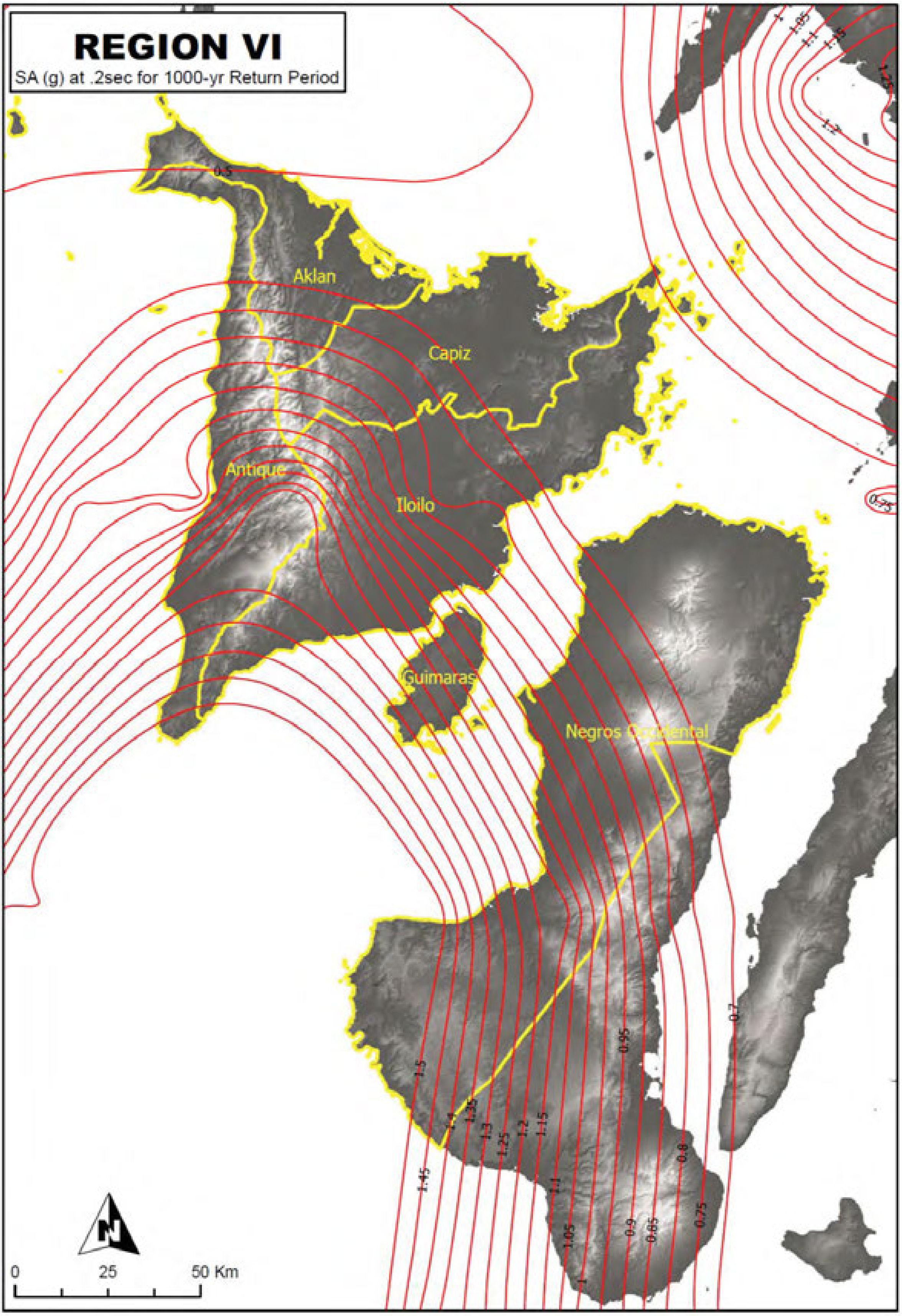


Figure 3B-28 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VI)

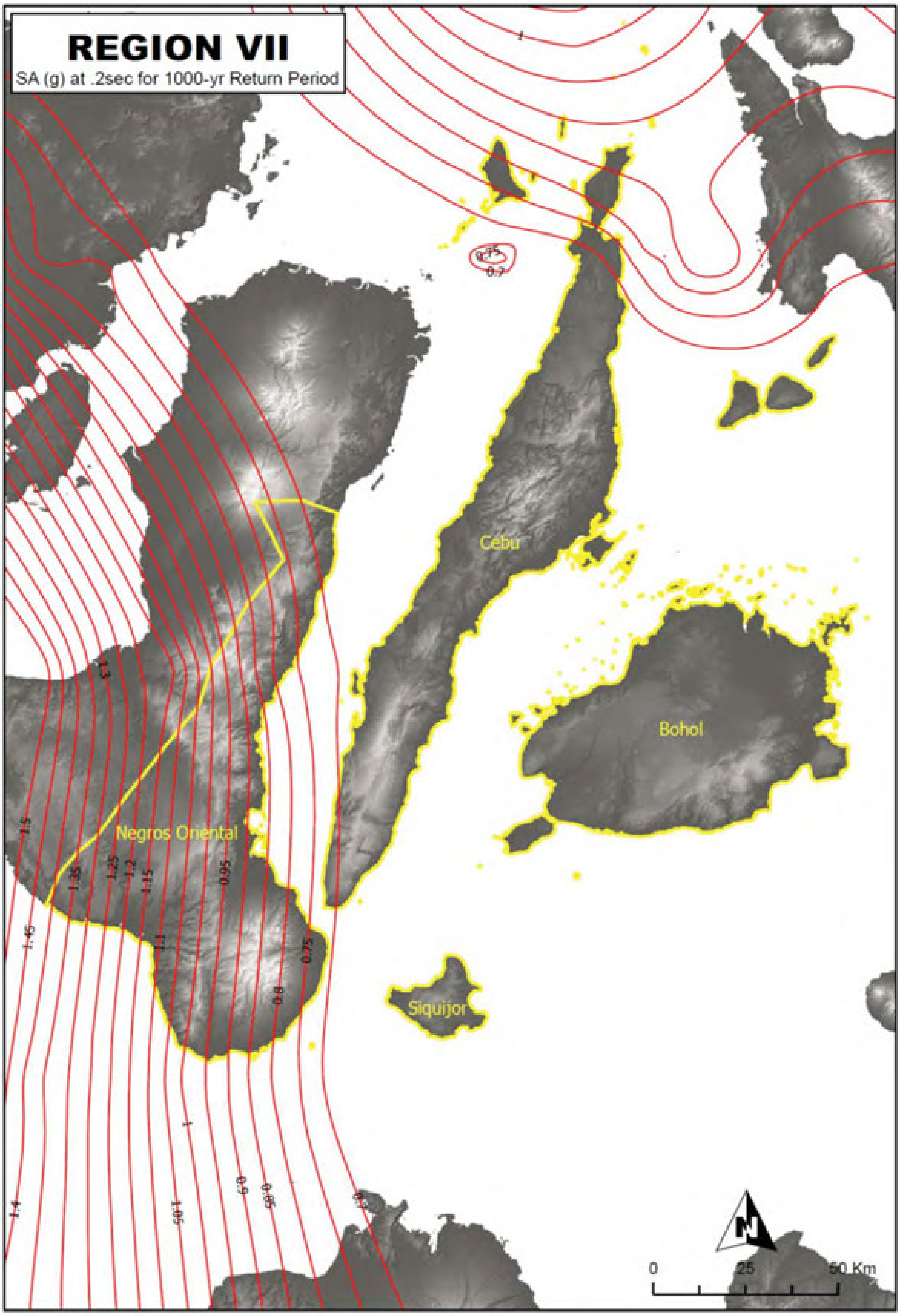


Figure 3B-29 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VII)

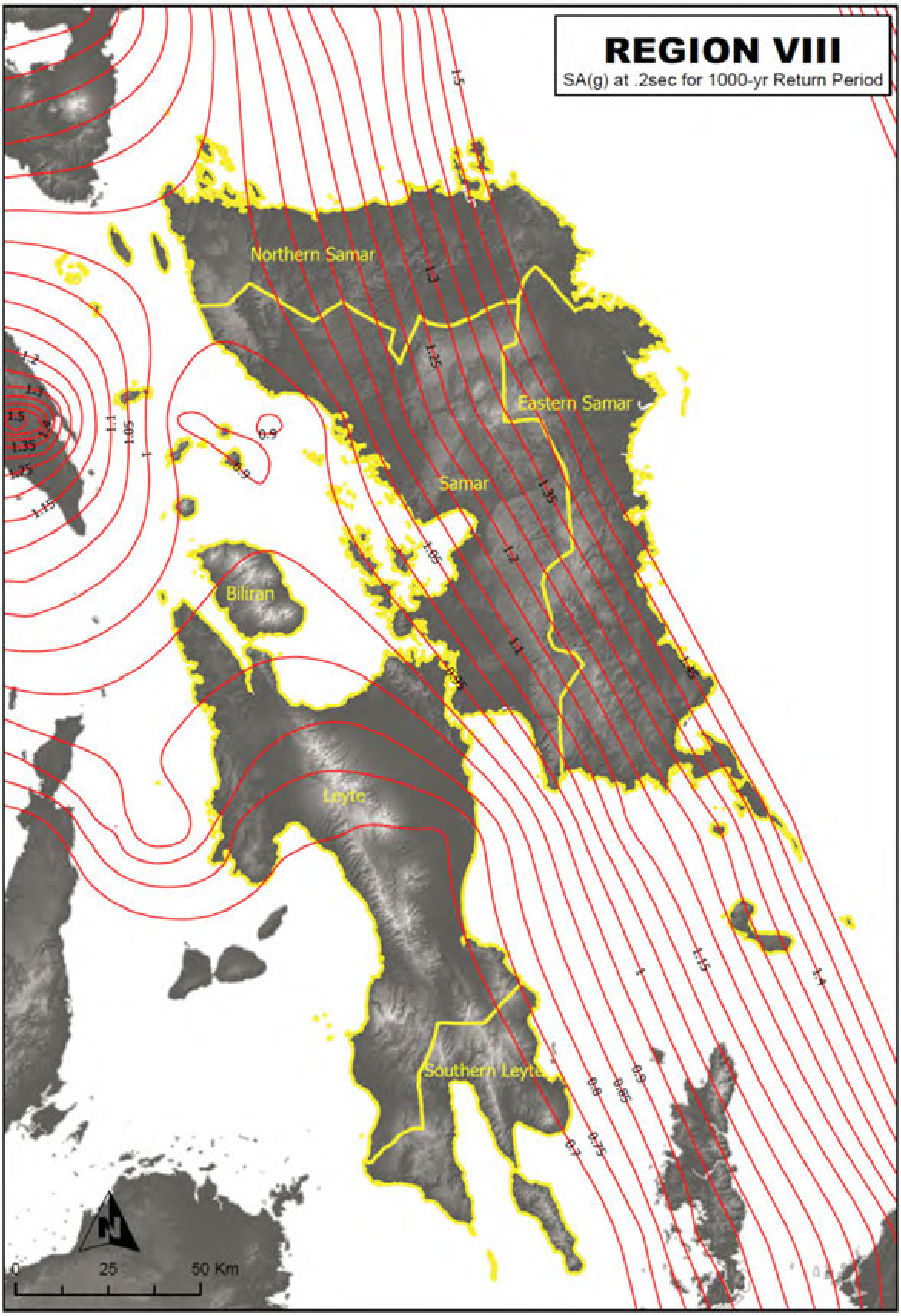


Figure 3B-30 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VIII)

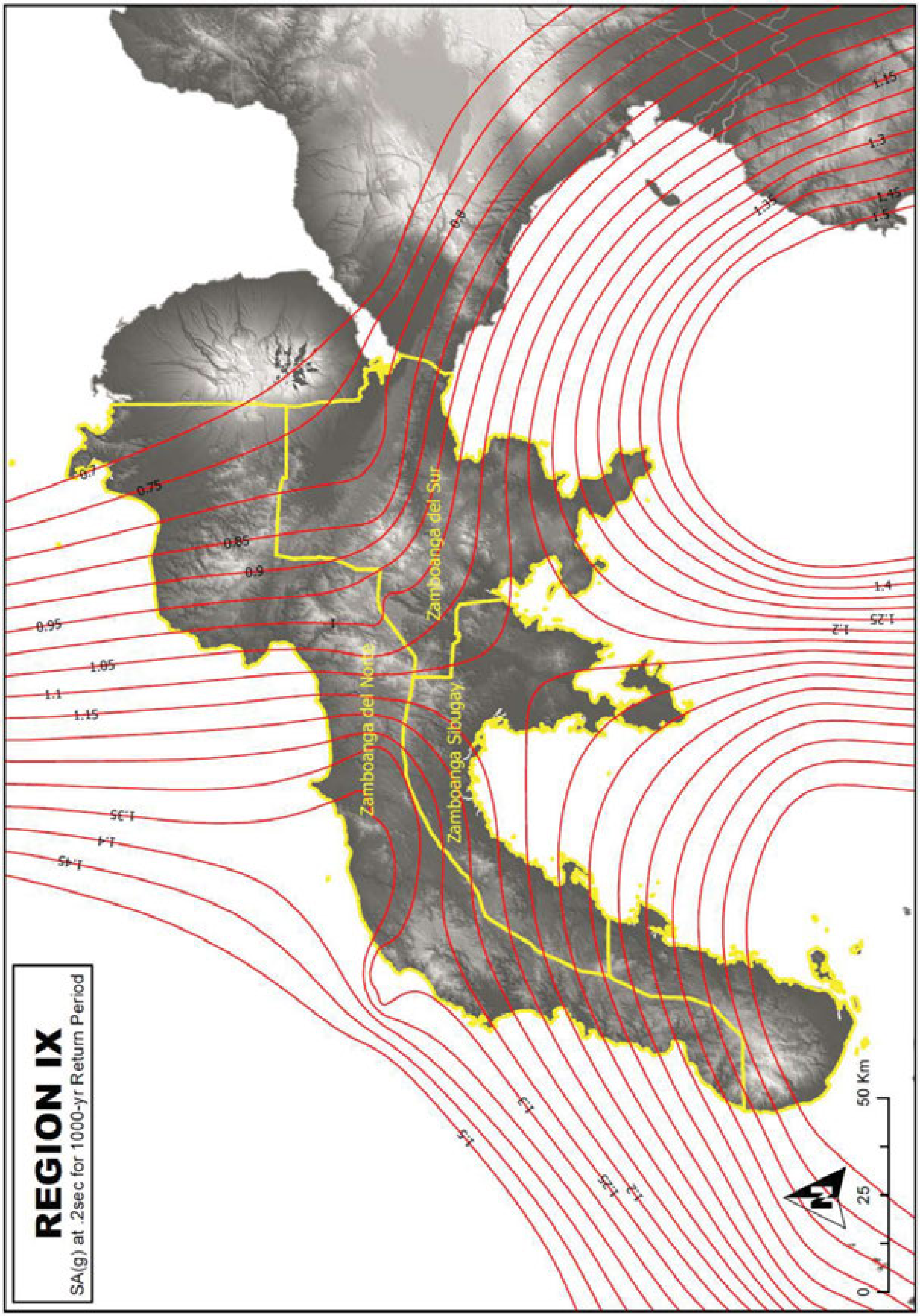


Figure 3B-31 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IX)

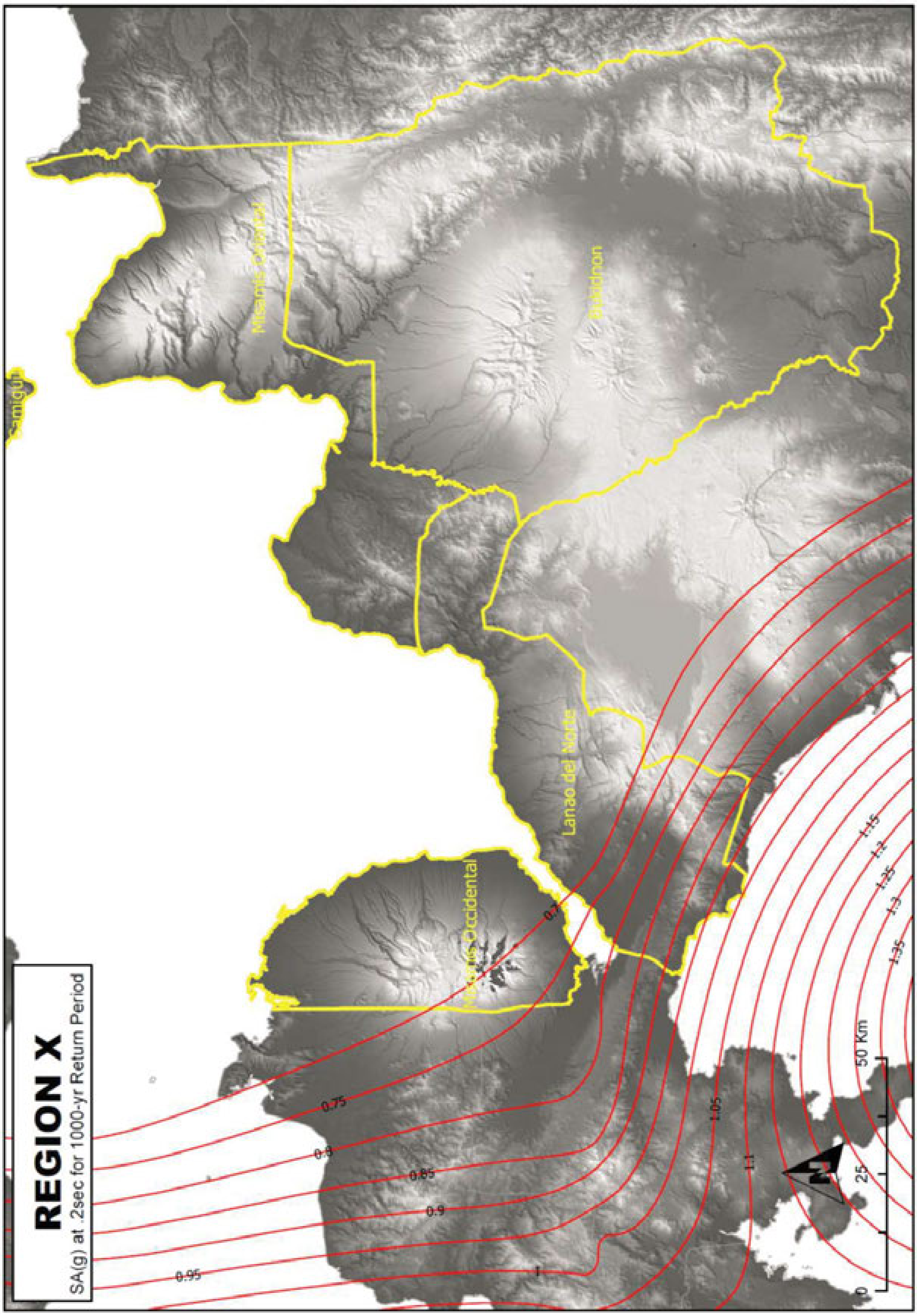


Figure 3B-32 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region X)

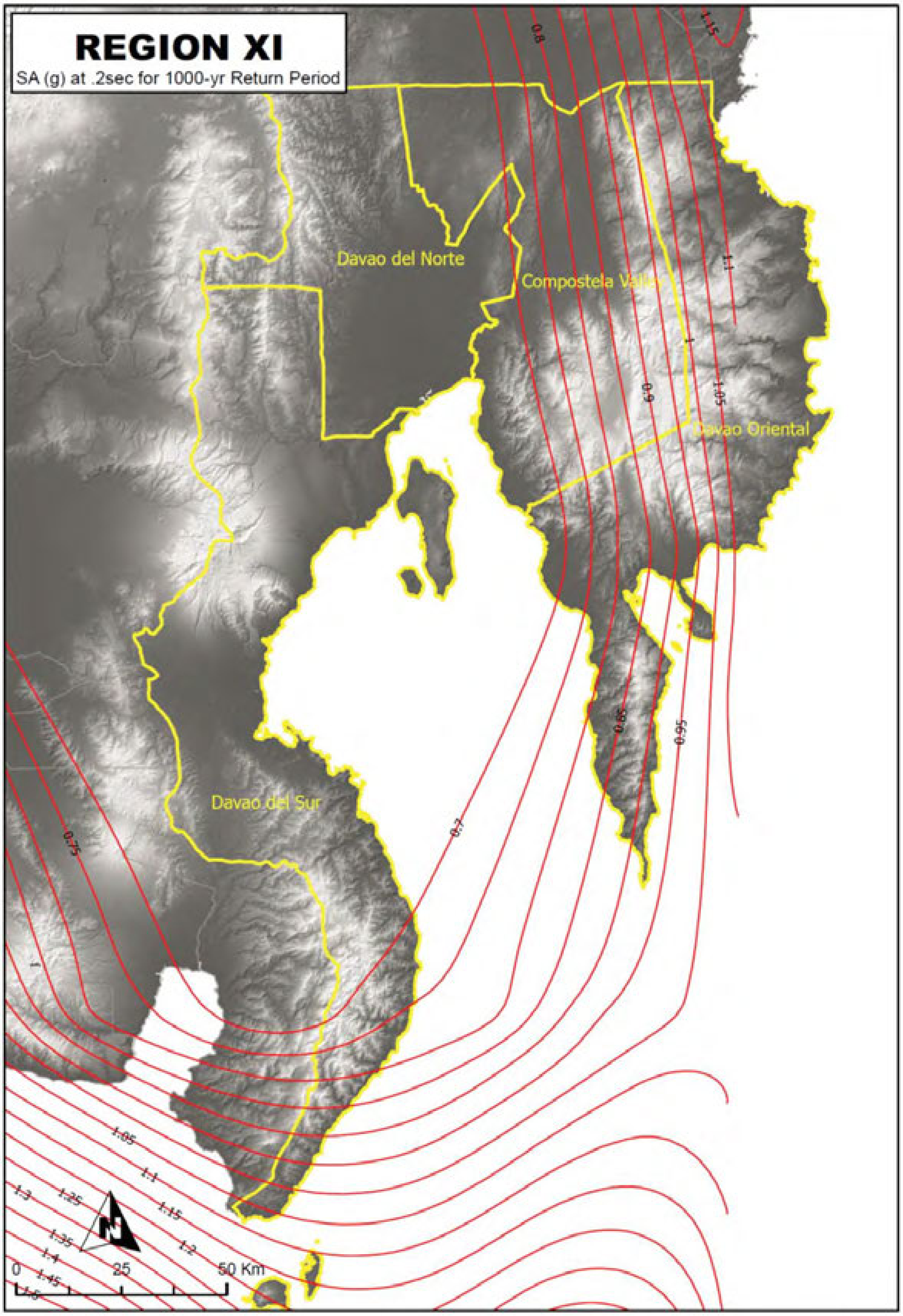


Figure 3B-33 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XI)

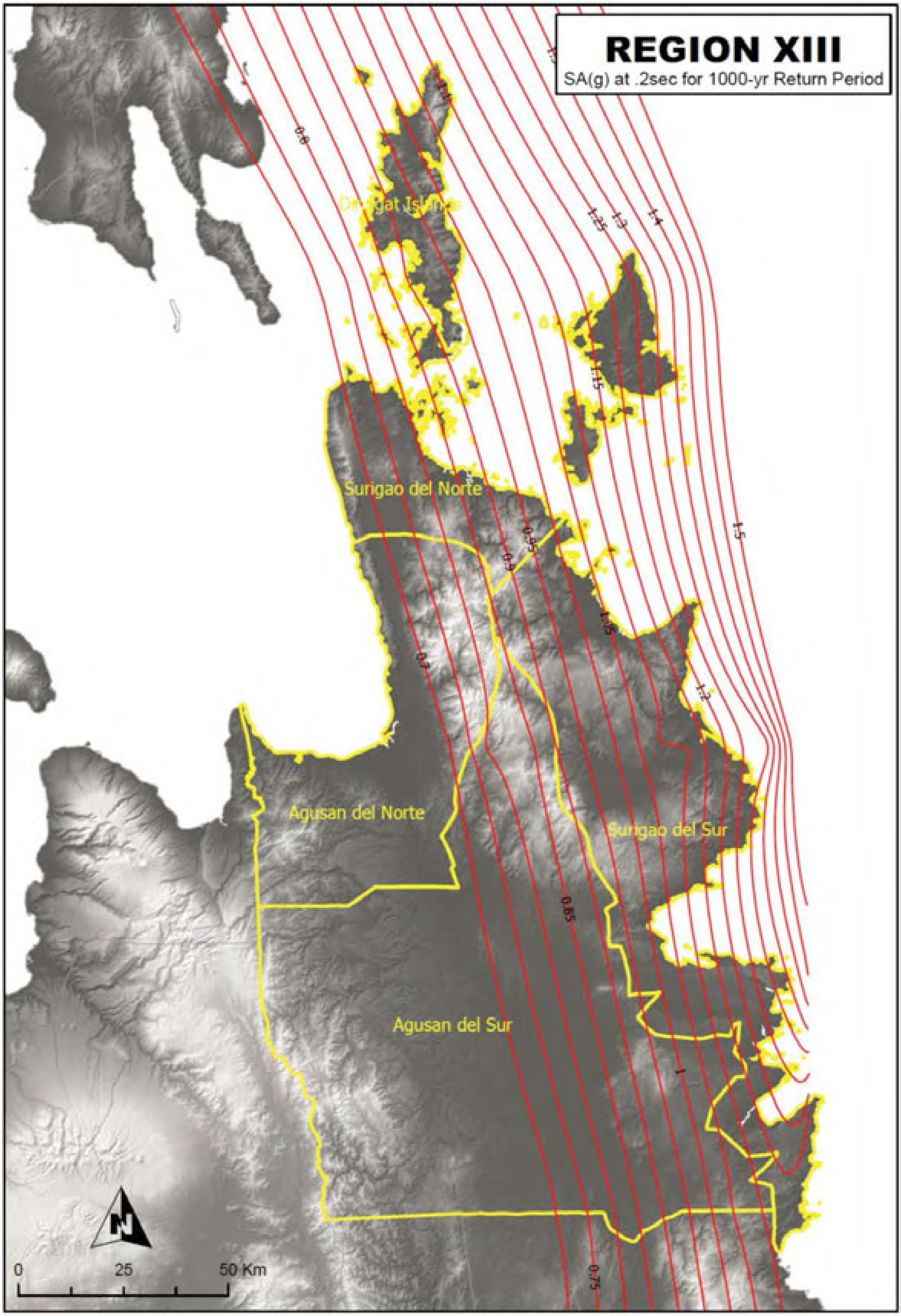


Figure 3B-34 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XIII)

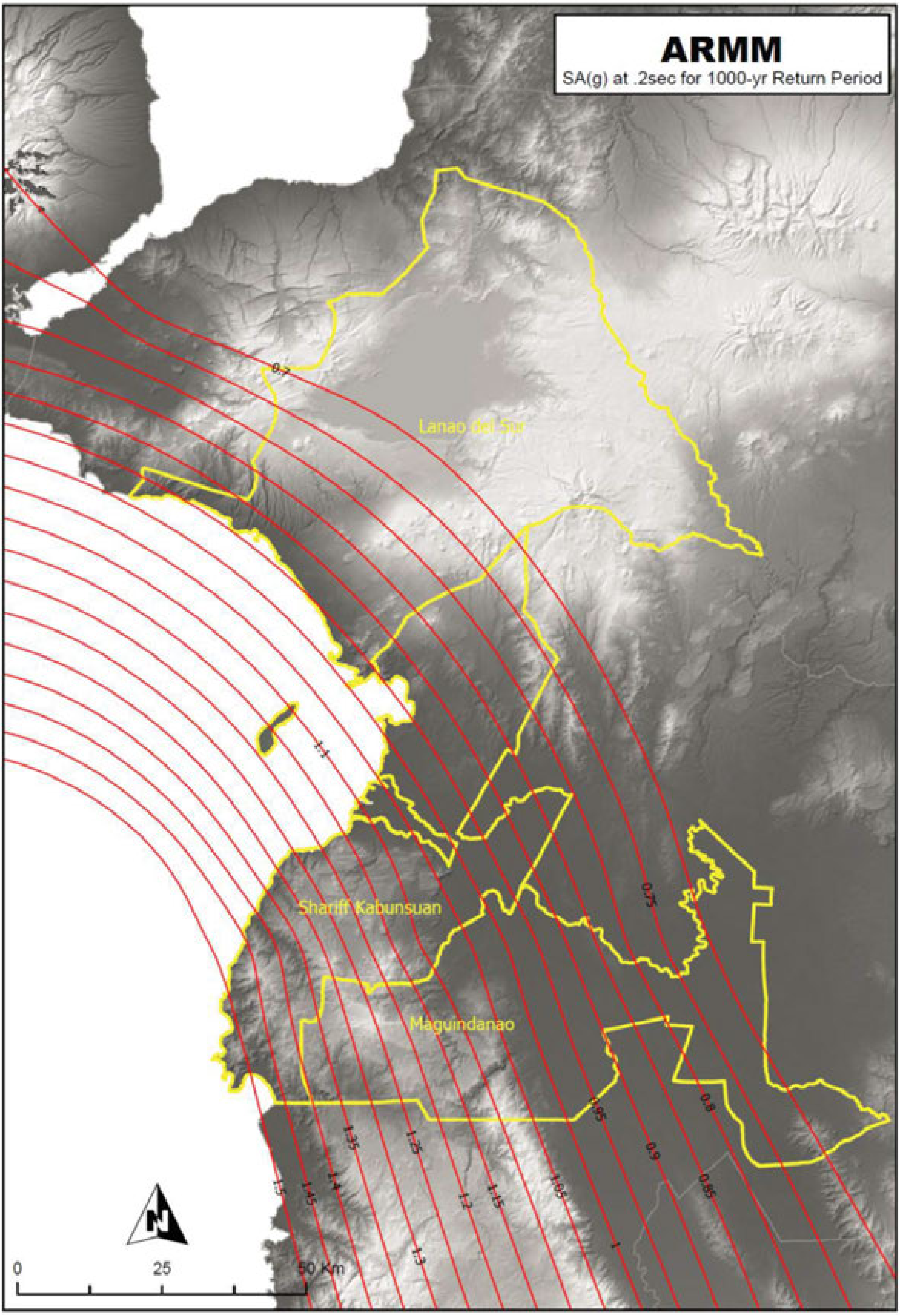


Figure 3B-35 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region ARMM)

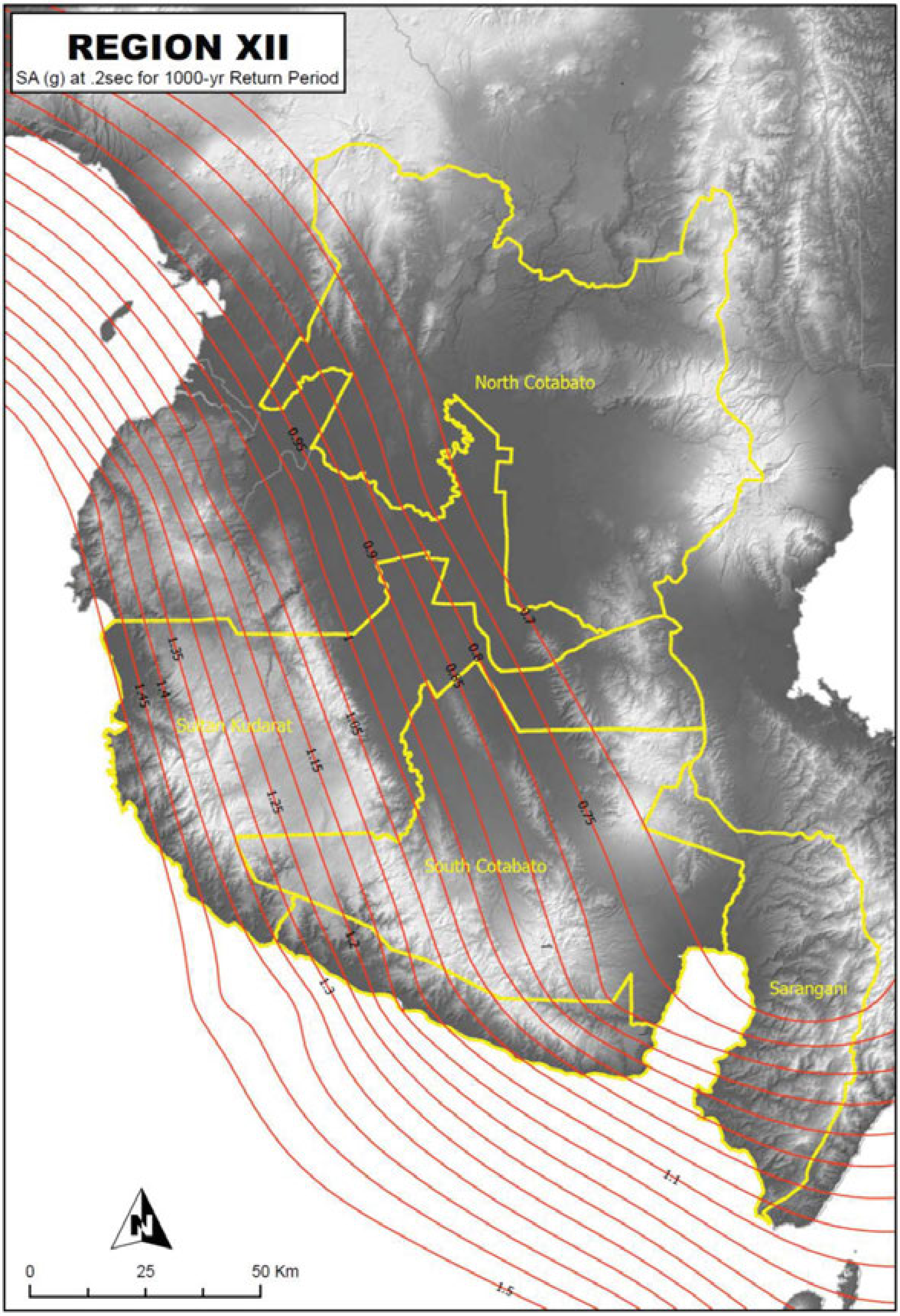


Figure 3B-36 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XII)

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT 1.0-SEC PERIOD

(1,000-YEAR RETURN PERIOD)

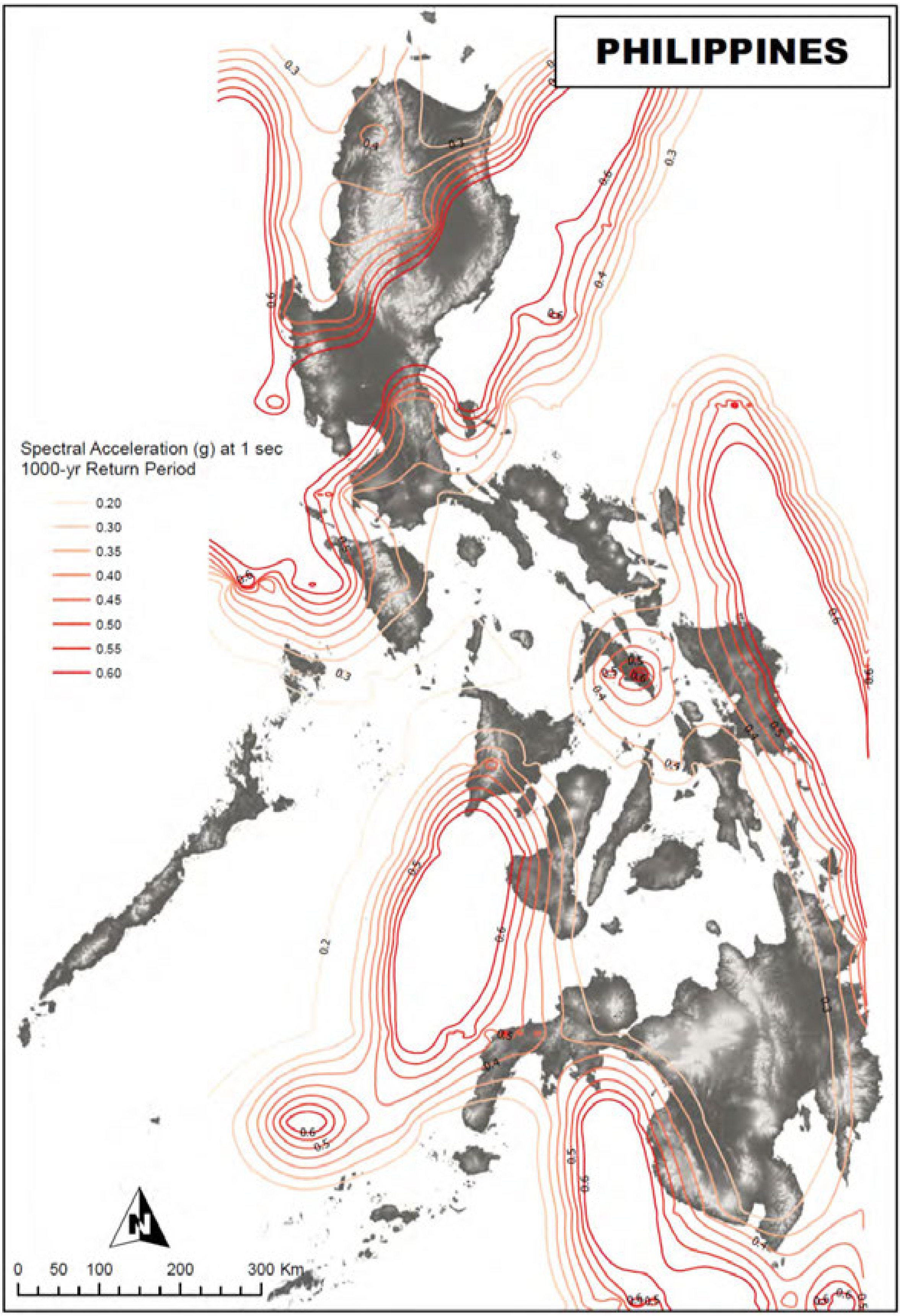


Figure 3B-37 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Entire Country)

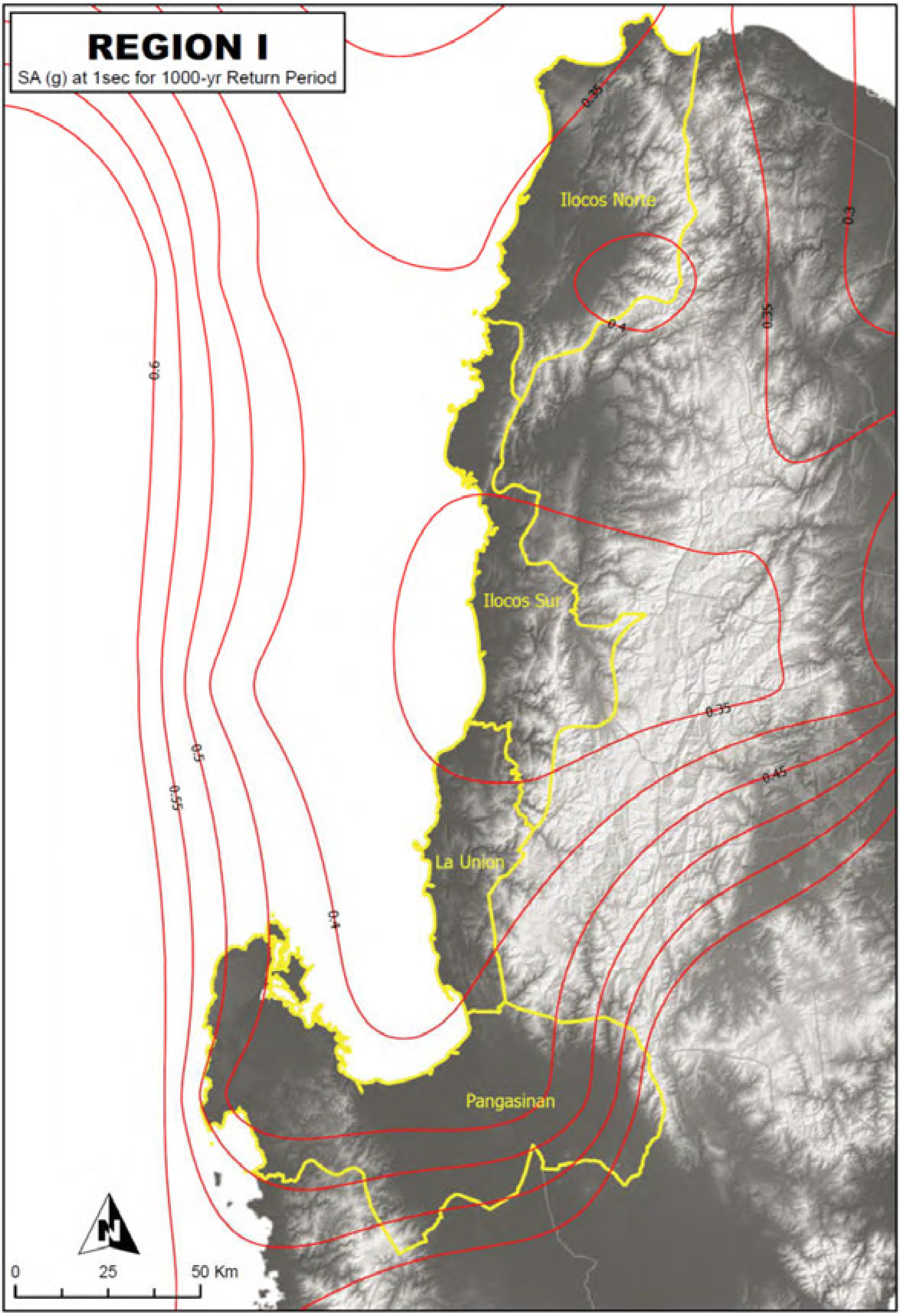


Figure 3B-38 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) Level 1 Earthquake Ground Motion (Region I)

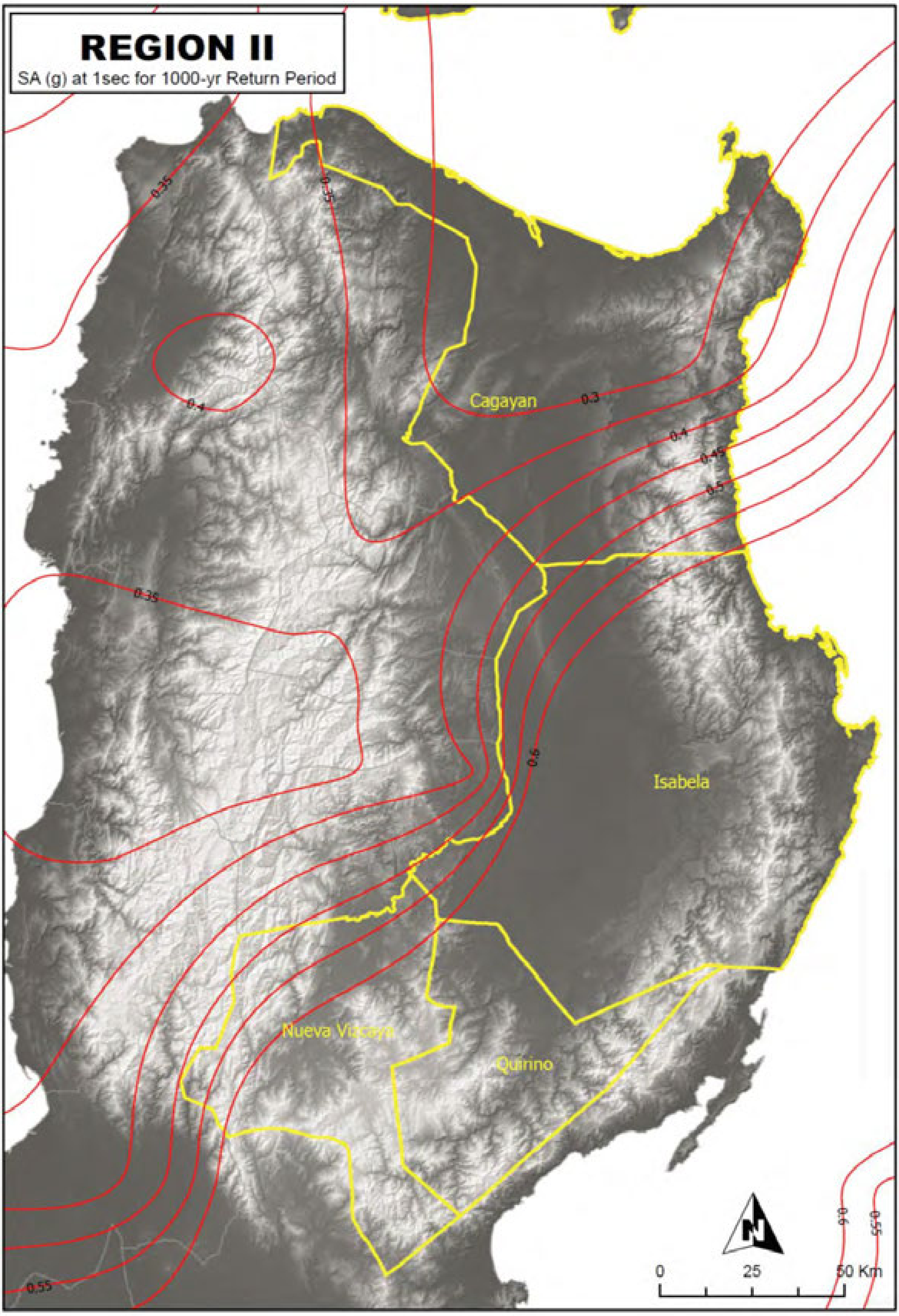


Figure 3B-39 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region II)

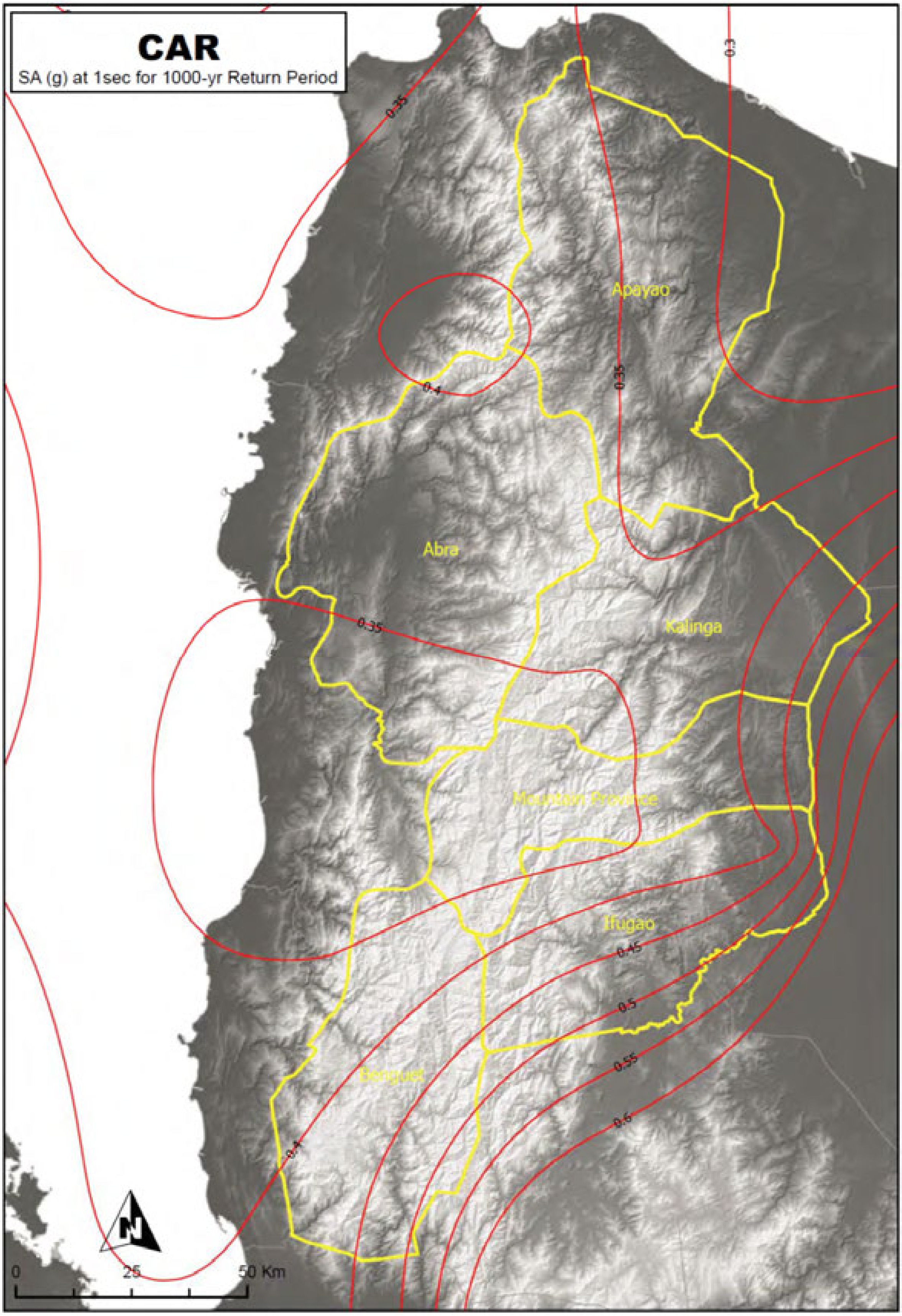


Figure 3B-40 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (CAR)

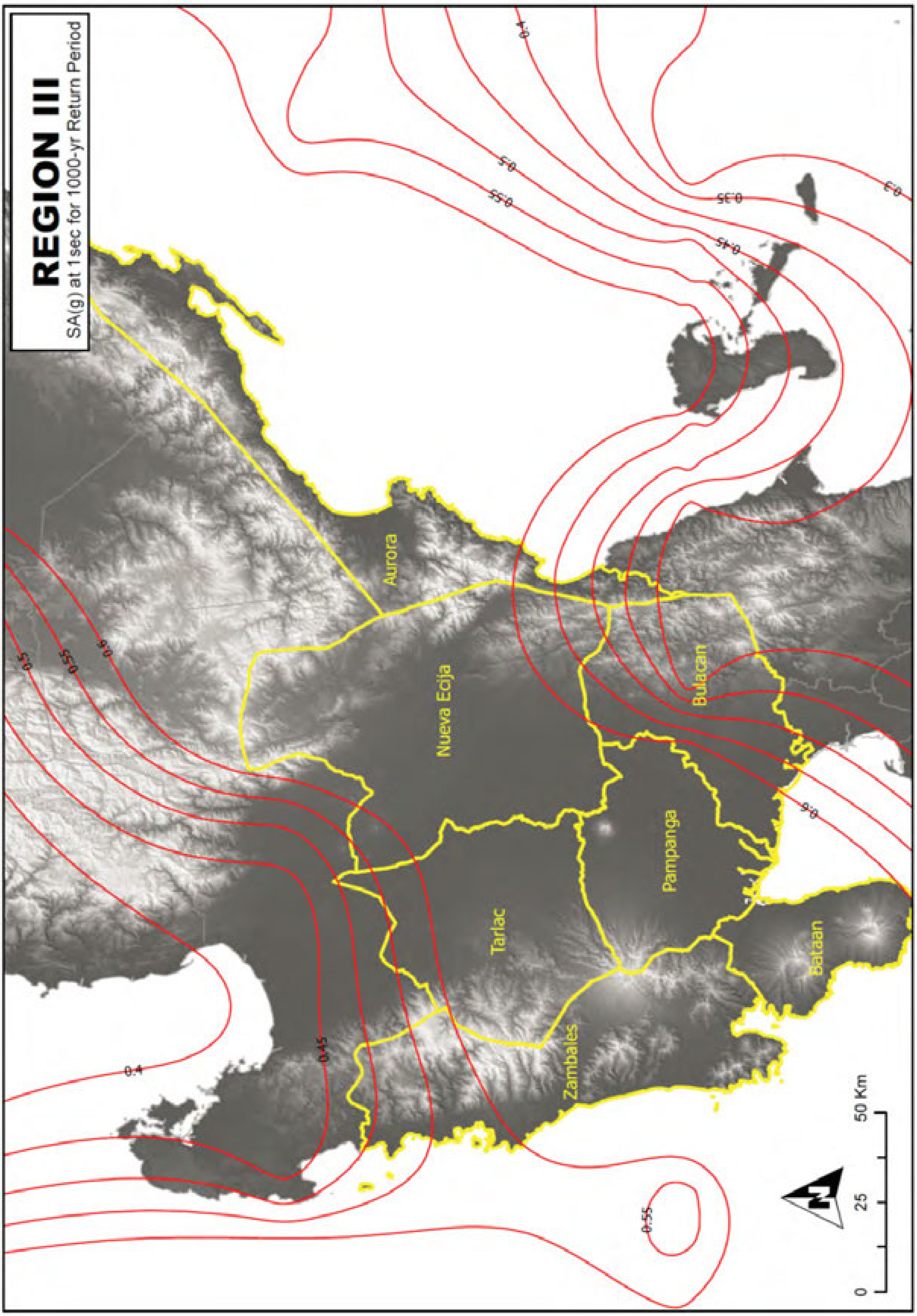


Figure 3B-41 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region III)

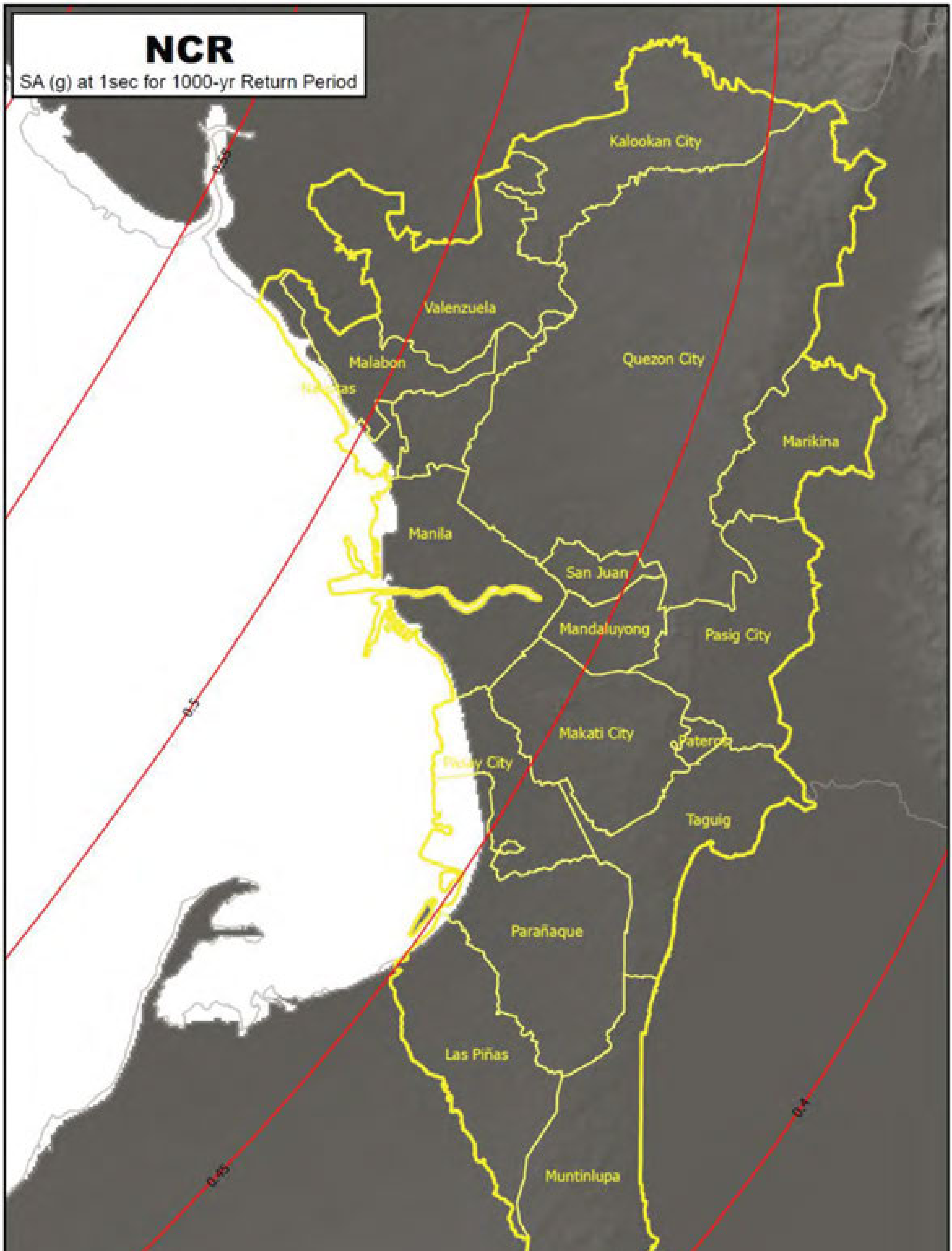


Figure 3B-42 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (NCR)

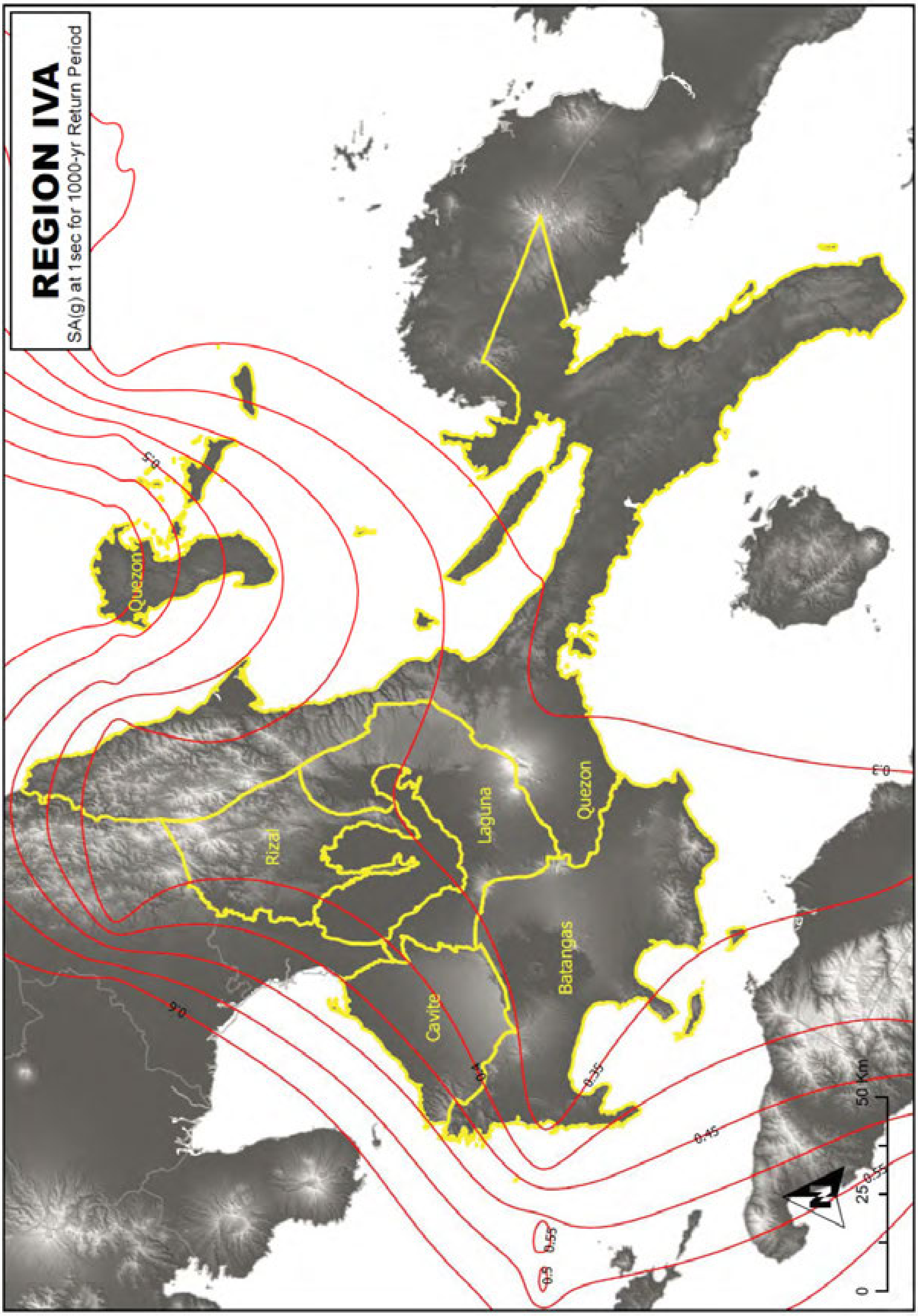


Figure 3B-43 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IV-A)

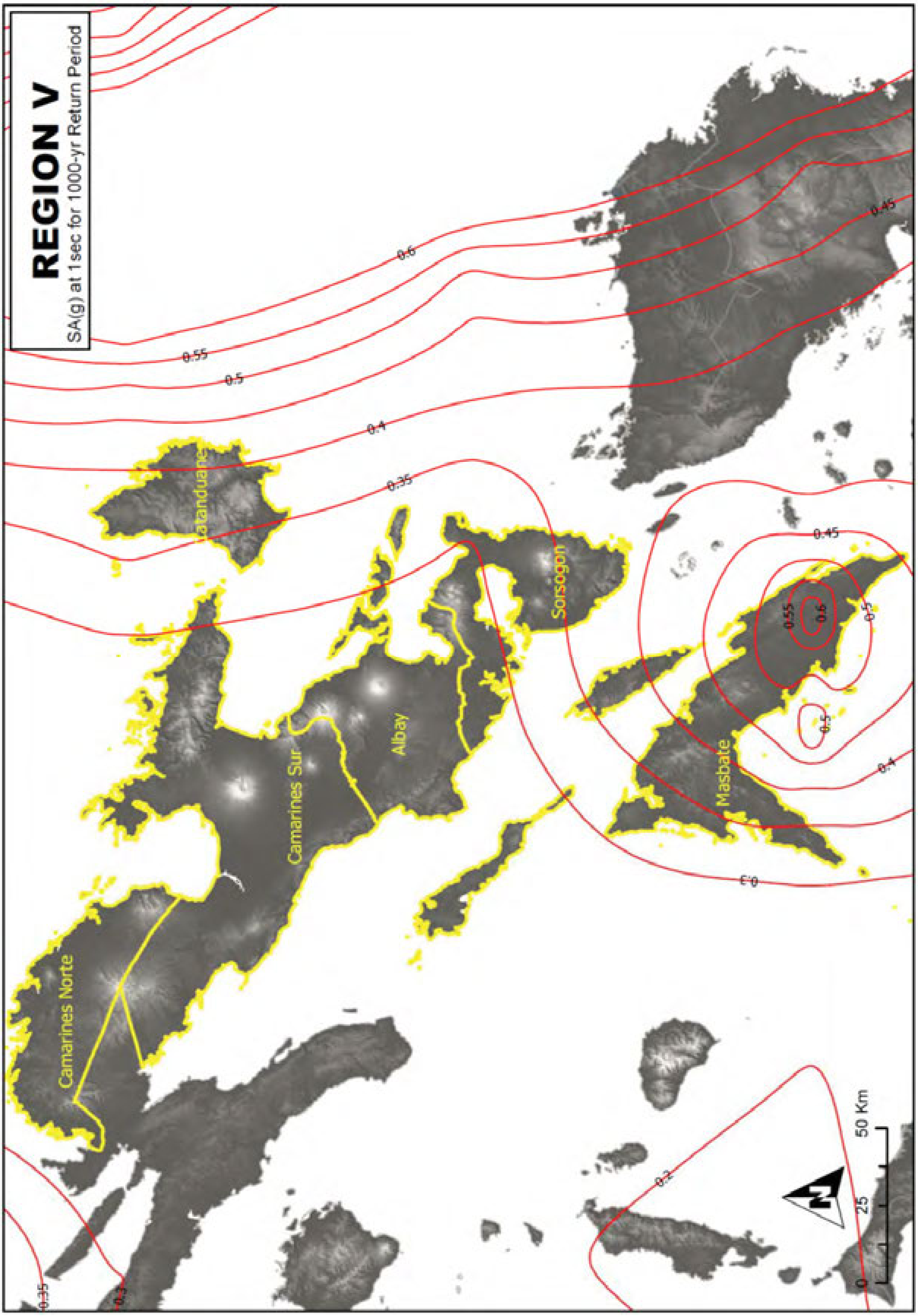


Figure 3B-44 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region V)

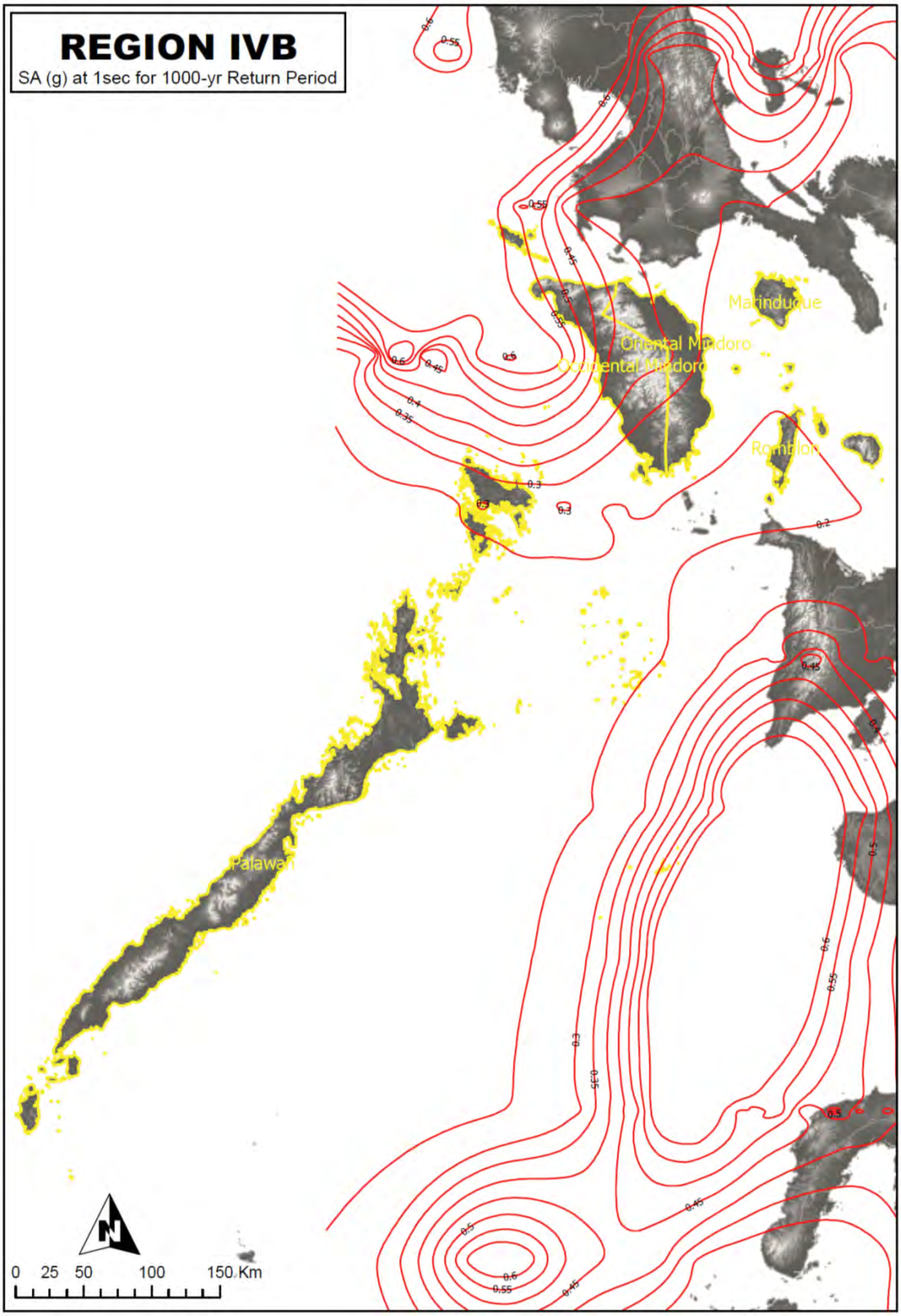


Figure 3B-45 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IV-B)

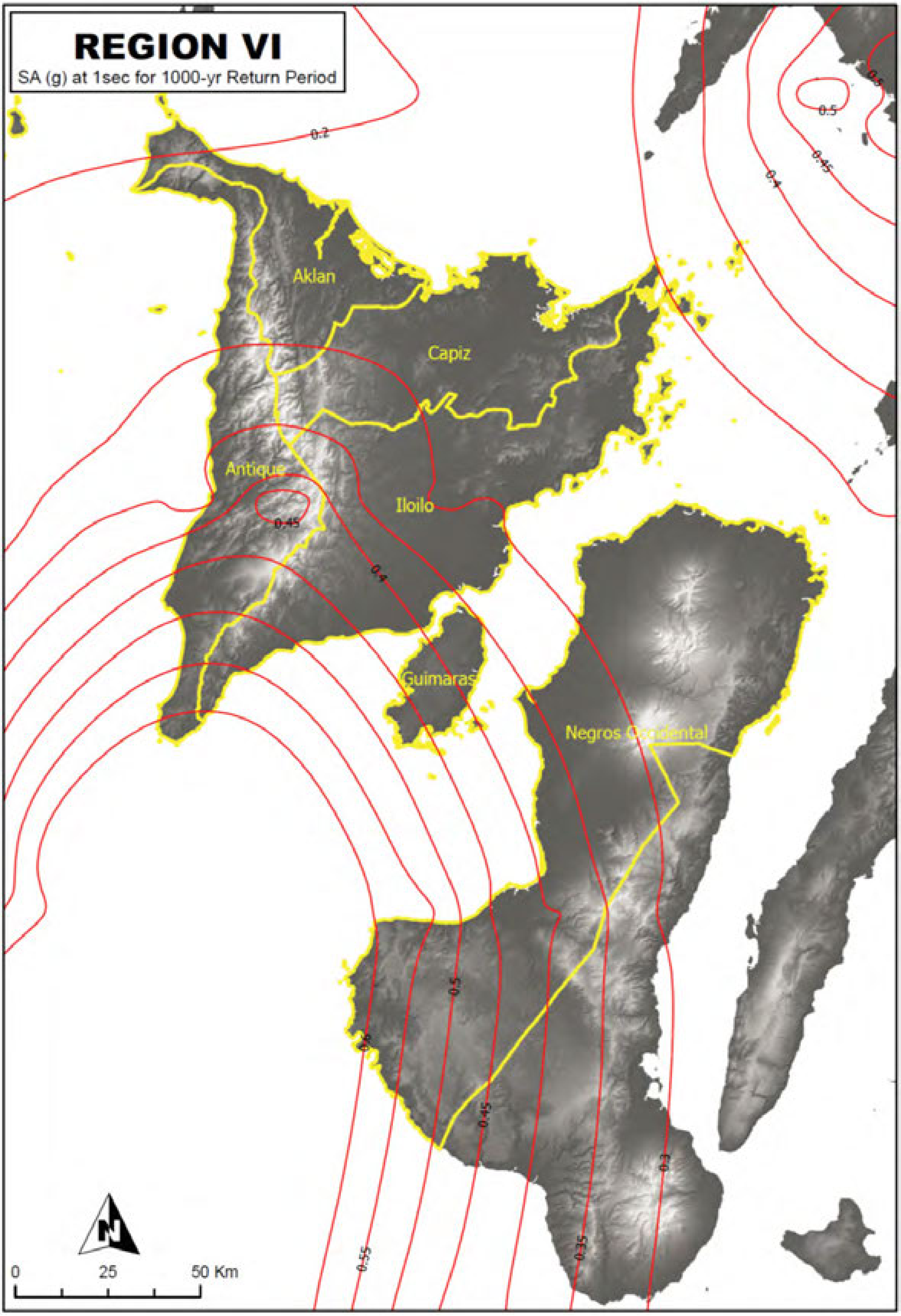


Figure 3B-46 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VI)

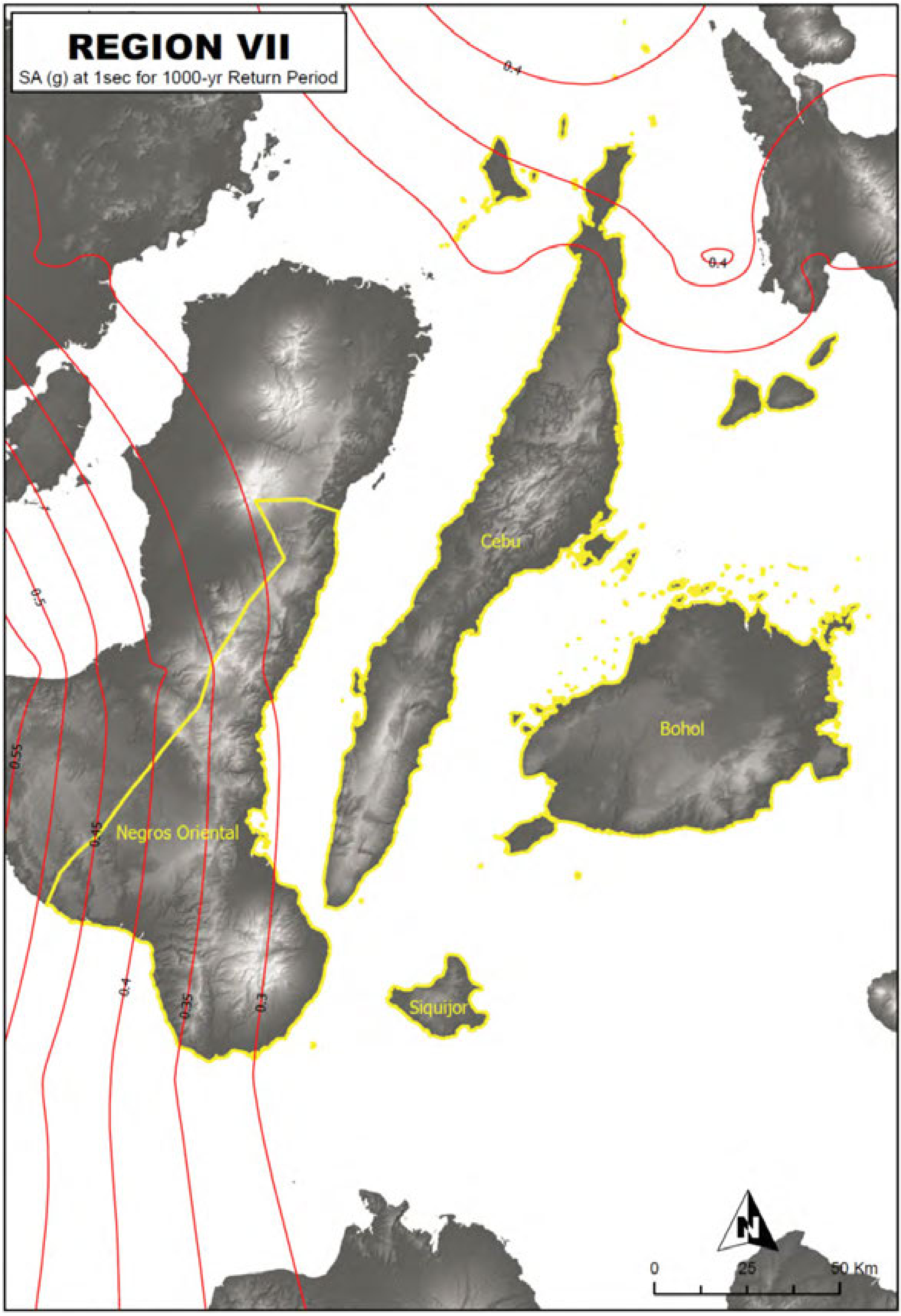


Figure 3B-47 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VII)

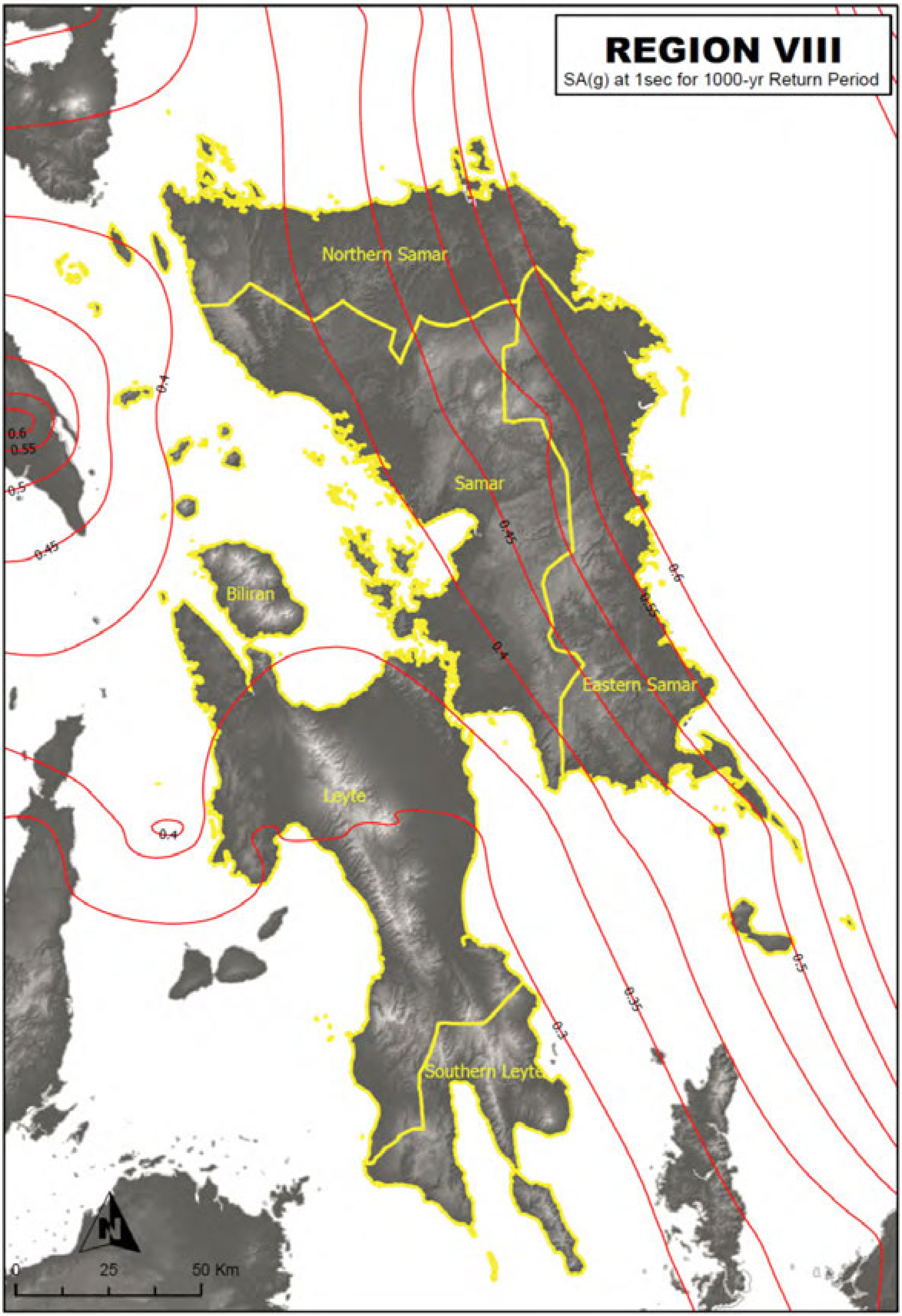


Figure 3B-48 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VIII)

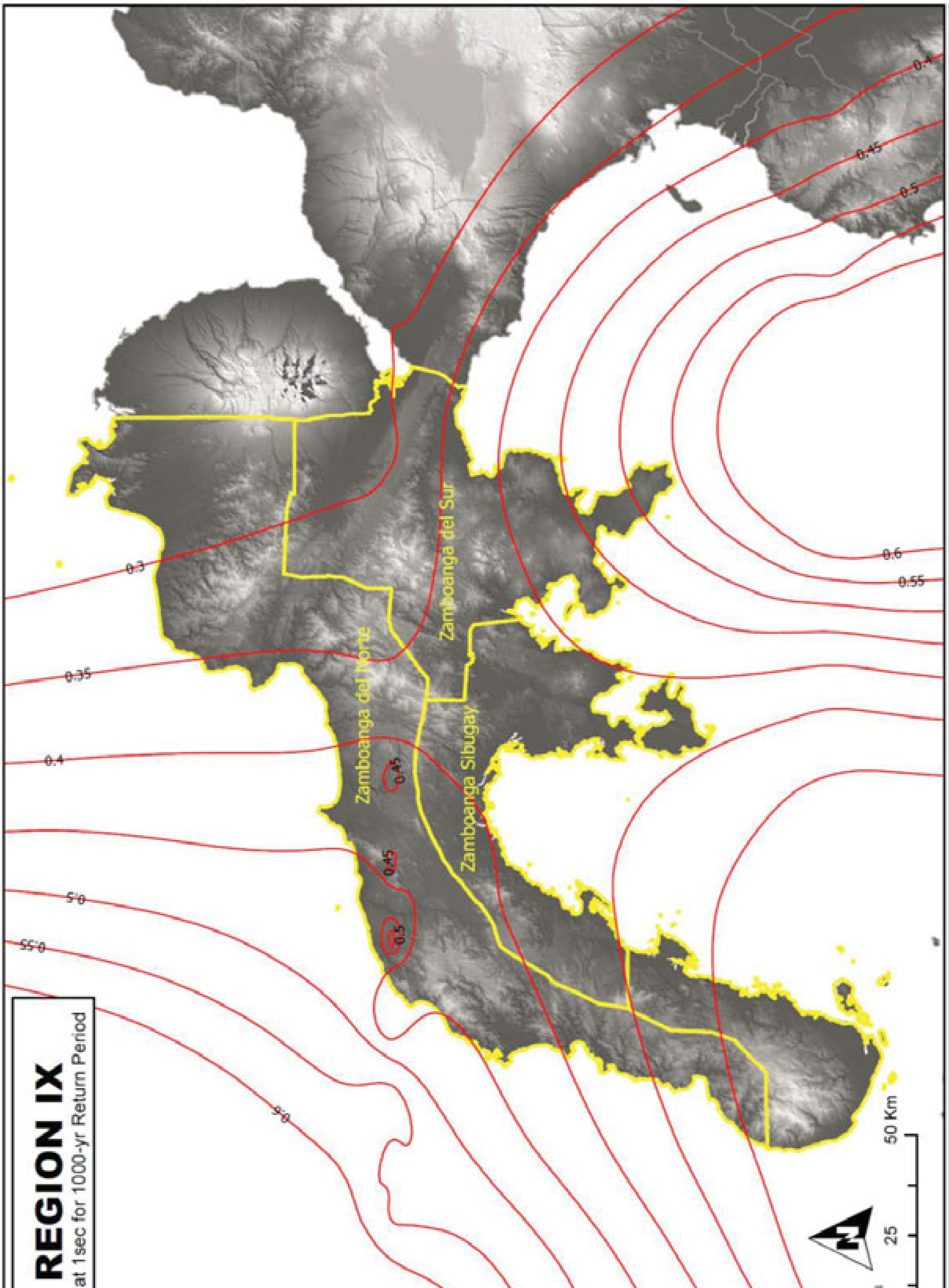


Figure 3B-49 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IX)

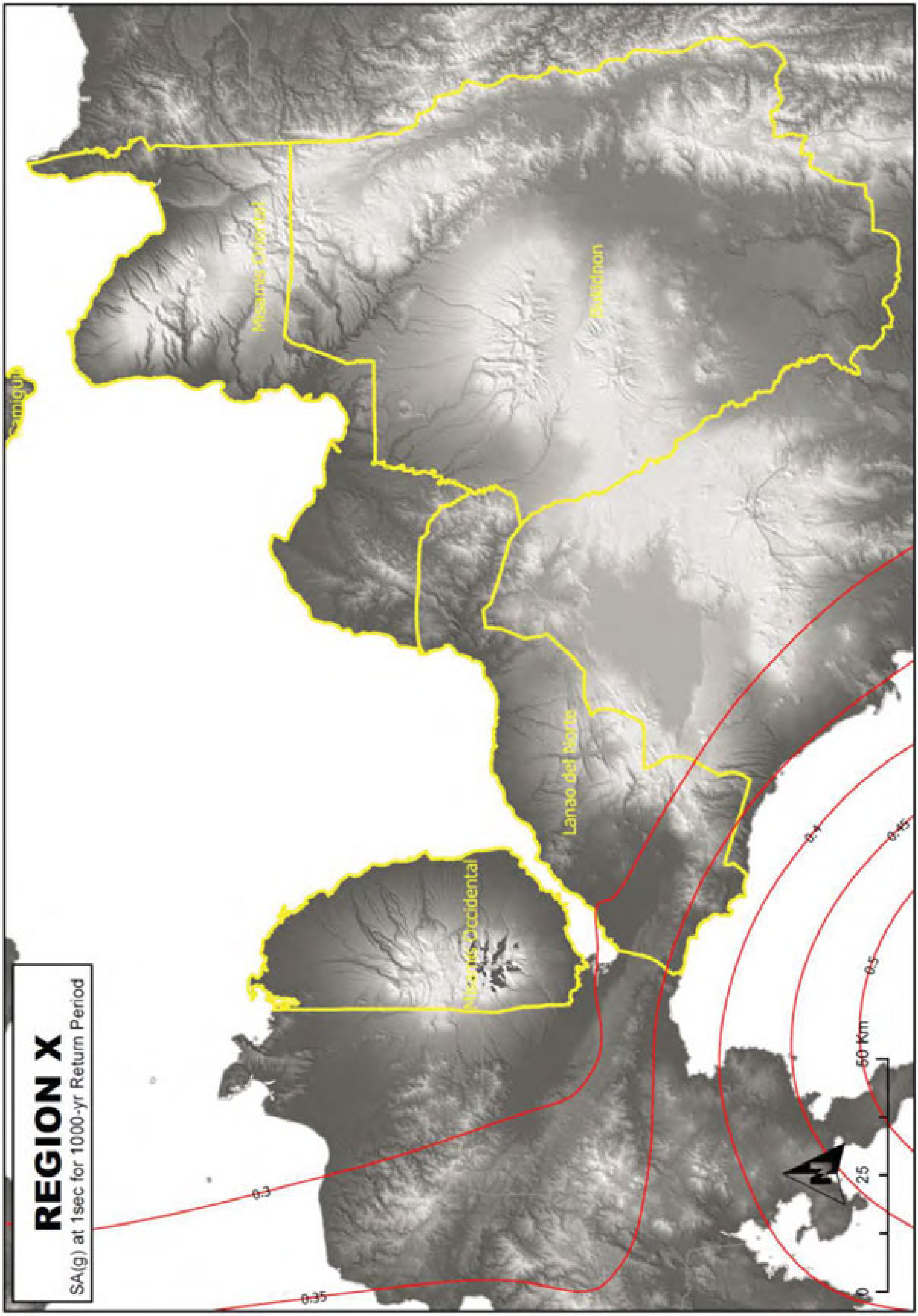


Figure 3B-50 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region X)

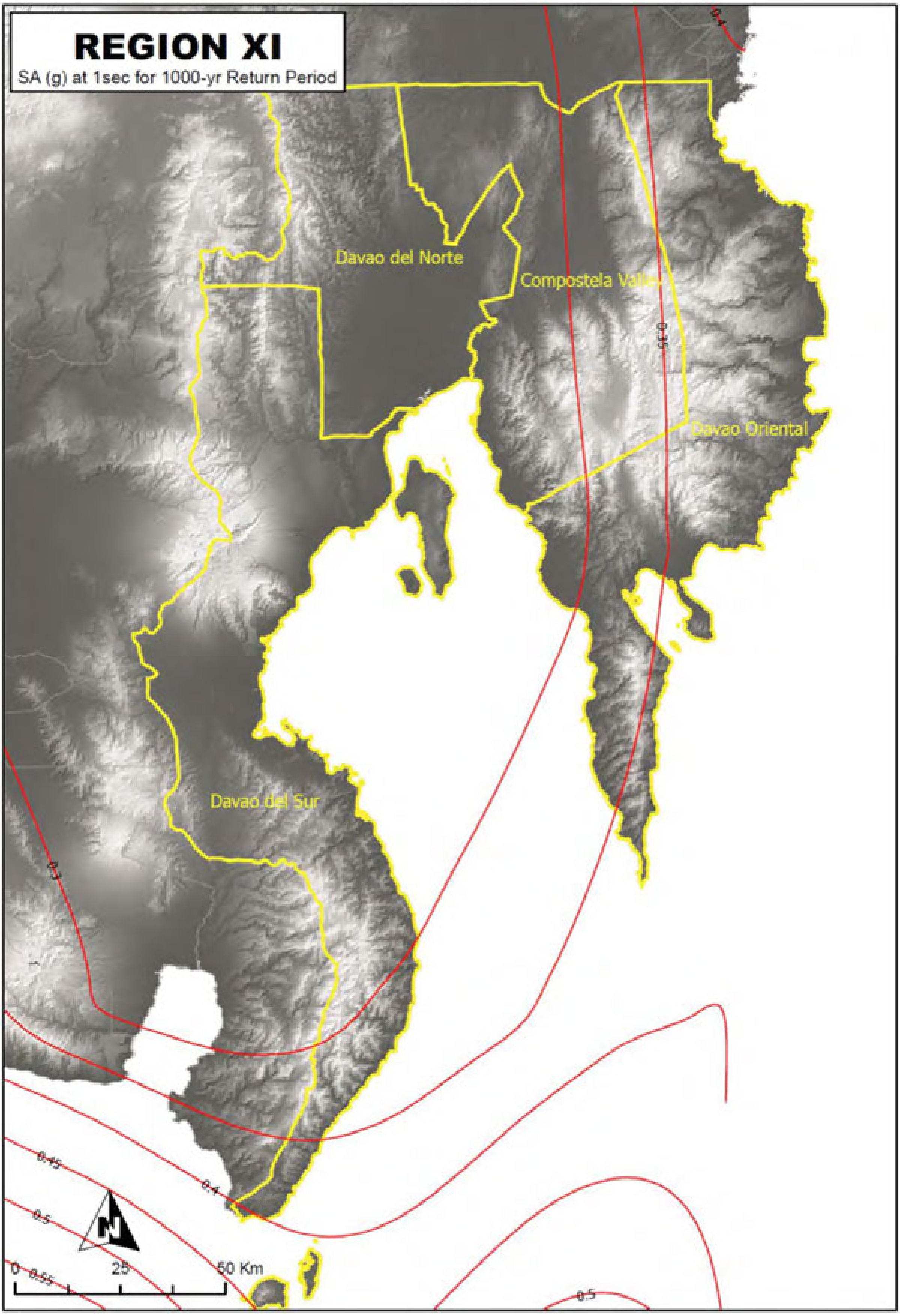


Figure 3B-51 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XI)

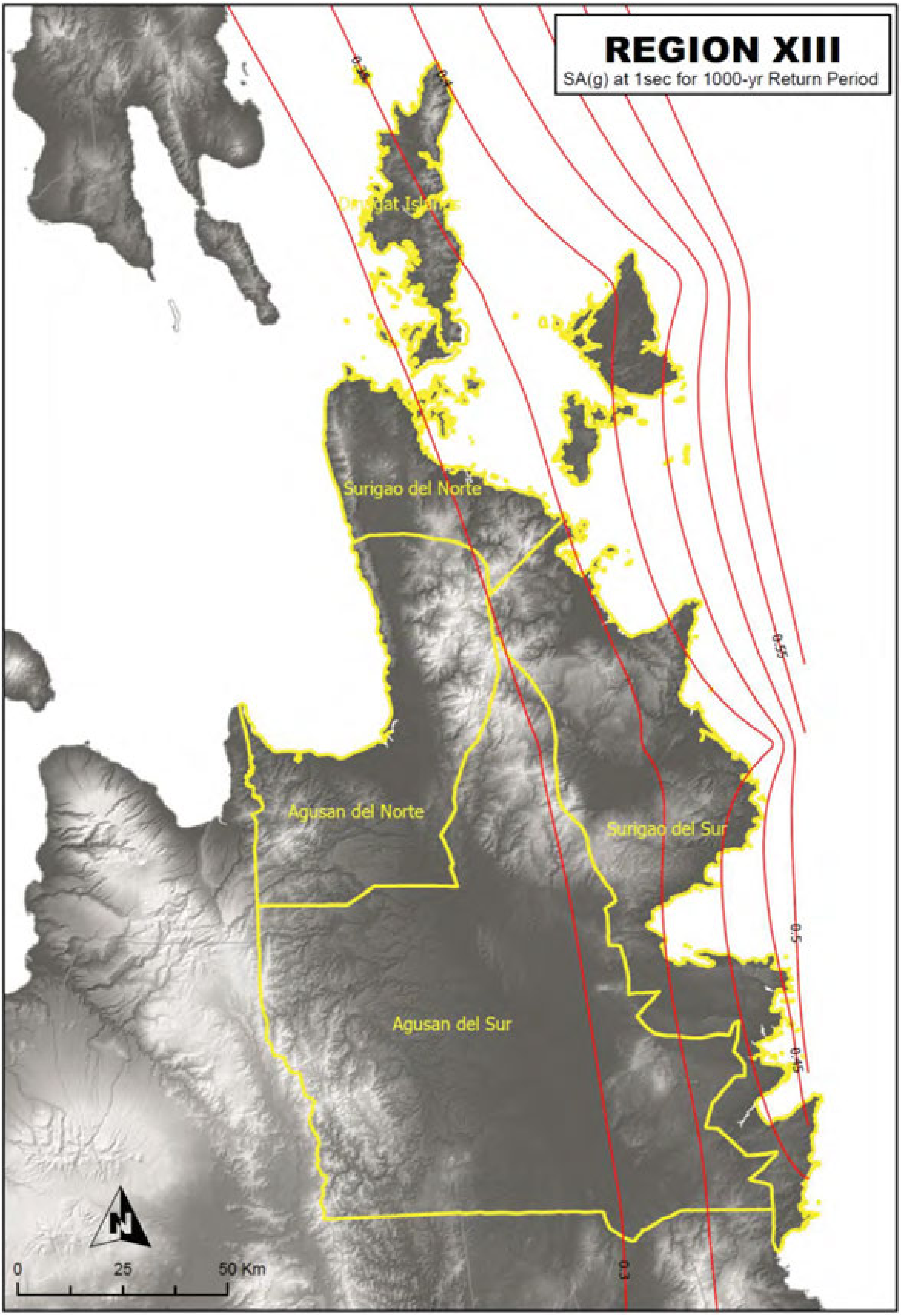


Figure 3B-52 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XIII)

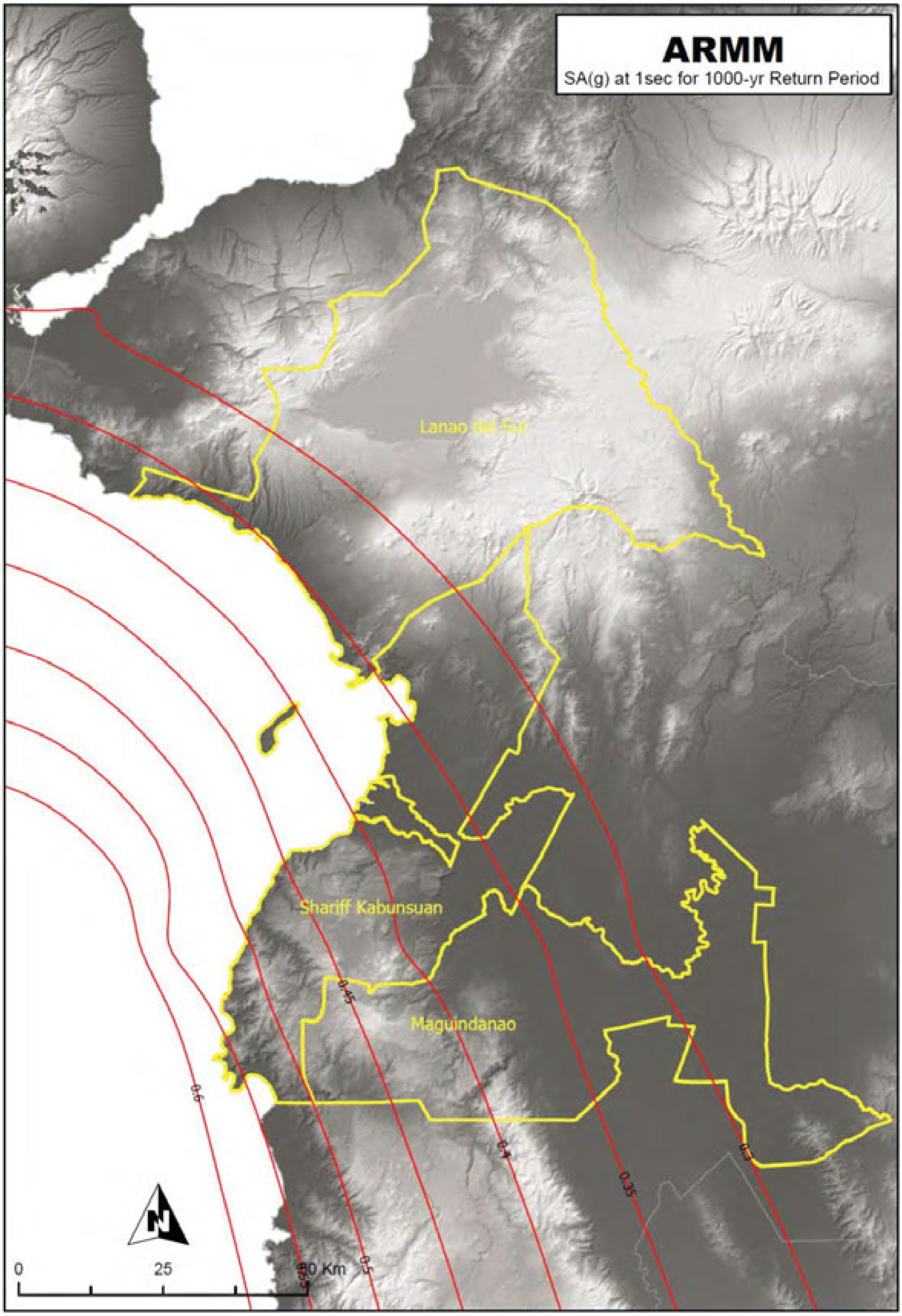


Figure 3B-53 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region ARMM)

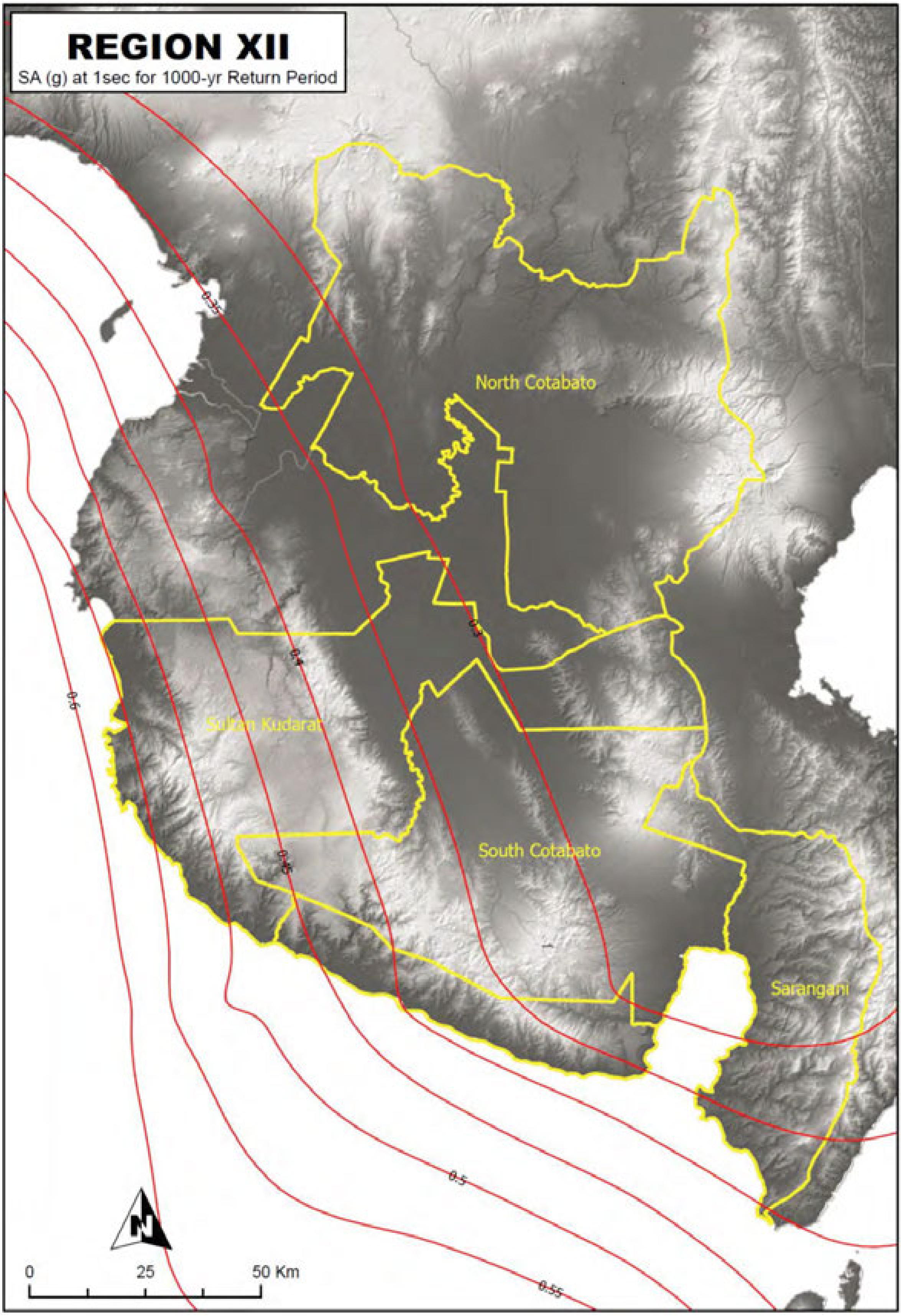


Figure 3B-54 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XII)

SECTION 4:

ANALYSIS REQUIREMENTS

4.1 GENERAL

(1) This section describes the analysis requirements for seismic effects on bridges under the extreme event limit state. Other analysis requirements not specified in this Section shall comply with the requirements of *Section 4: Structural Analysis and Evaluation, AASHTO LRFD Bridge Design Specifications (2012 or later)* concerning the methods of modeling and analysis for the design and evaluation of bridges. Likewise, the analysis for bridges shall be consistent with the updated and revised DPWH Guide Specifications.

(2) The minimum analysis requirements for seismic effects shall be as specified in Table 4.3.1-1 of this Section.

(3) For the modal methods of analysis, specified in Articles 4.3.2 and 4.3.3, the design response spectrum specified in Figure 3.6.1-1 and Equations 3.6.2-1, 3.6.2-3, and 3.6.2-4 of these Specifications shall be used.

(4) Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their operational classification and geometry. However, the minimum requirements, as specified in Articles 4.6 and 5.3 of these Specifications, shall apply.

(5) Any method of analysis that satisfies the requirements of equilibrium and compatibility and utilizes stress-strain relationships for the proposed materials may be used.

(6) The Designer shall be responsible for the implementation of computer programs used to facilitate structural analysis and for interpretation and use of results. The name, version, and release date of software used should be indicated in the design report and the contract documents.

Commentary C4.1

Many computer programs are available for bridge design incorporating various methods of analysis ranging from simple formula to detailed finite element procedures. Since computer programs have specific engineering assumptions embedded in the code, which may or may not be applicable to a particular case, the Designer should clearly understand the basic assumptions and limitations as well as the methodology implemented in the program.

A computer program is only a tool, and the user is responsible for the generated output and results that should be verified to the extent possible.

Computer programs should be verified against the results of:

- Universally accepted closed-form solutions,
- Other previously verified computer programs, or
- Physical testing.

The purpose of identifying the software is to establish code compliance and to provide a means of locating bridges deigned with software that may later be found deficient.

4.2 SINGLE-SPAN BRIDGES

(1) The seismic analysis and design for abutments of single-span bridges shall comply with the requirements of *Section 11 (Walls, Abutments and Piers) of the AASHTO LRFD Bridge Design Specifications 2012 (or later versions)*.

(2) Connections between the bridge superstructure and the abutments shall be designed for the minimum force requirements as specified in Article 5.3 of these Specifications.

(3) Minimum support length requirements shall be satisfied at each abutment as specified in Article 4.6 of this Section.

Commentary C4.2

A single-span bridge is comprised of a superstructure unit supported by two abutments with no intermediate piers.

4.3 MULTISPAN BRIDGES

4.3.1 Selection of Method

(1) For multi-span structures, the minimum analysis requirements shall be as specified in Table 4.3.1-1 in which:

*	=	No seismic analysis required	MM	=	Multimode elastic method
UL	=	Uniform load elastic method	TH	=	Time history method
SM	=	Single-mode elastic method			

Table 4.3.1-1—Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1	As specified in Article 4.2	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	MM/TH
4		SM/UL	MM	MM	MM	MM/TH	MM/TH

Time history analysis may be required by the DPWH on important and complicated bridges.

(2) Except as specified below, bridges satisfying the requirements of Table 4.3.1-2 may be taken as "Regular" bridges. Bridges not satisfying the requirements of Table 4.3.1-2 shall be taken as "Irregular" bridges.

(3) Curved bridges comprised of multiple simple-spans shall be considered to be "Irregular" if the subtended angle in plan is greater than 20 degrees. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

(4) A curved continuous-girder bridge may be analyzed as if it were straight, provided all of the following requirements are satisfied:

- The bridge is "Regular" as defined in Table 4.3.1-2, except that for a two-span bridge the maximum span length ratio from span to span must not exceed 2;
- The subtended angle in plan is not greater than 90 degrees; and
- The span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

If these requirements are not satisfied, then curved continuous-girder bridges must be analyzed using the actual curved geometry.

Table 4.3.1-2—Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	-	4	4	3	2

Commentary C4.3.1

(1) The selection of the method of analysis depends on seismic zone, regularity, and operational classification of the bridge.

(2) Regularity is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry; and no large changes in these parameters from span to span or support-to-support, abutments excluded. A more rigorous analysis procedure may be used in lieu of the recommended minimum.

4.3.2 Single-Mode Method of Analysis

4.3.2.1 General

Either of the two single-mode methods of analysis specified herein may be used where appropriate.

4.3.2.2 Single-Mode Spectral Method

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction, For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, specified in Article 3.6.2 of these Specifications, and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

Commentary C4.3.2.2

(1) The single-mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustrating its application are given in AASHTO (1983) and ATC (1981).

- Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o, as shown in Figure C4.3.2.2-1:

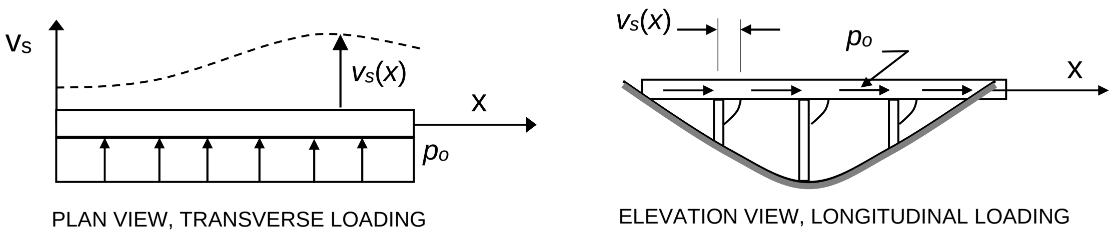


Figure C4.3.2.2-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

- Calculate factors α, β and γ as:

$$\alpha = \int v_s(x)dx \qquad \text{(C4.3.2.2-1)}$$

$$\beta = \int w(x)v_s(x)dx \qquad \text{(C4.3.2.2-2)}$$

$$\gamma = \int w(x)v_s^2(x)dx \qquad \text{(C4.3.2.2-3)}$$

where:

p_o = Uniform load arbitrarily set equal to 1.0, (N/m)

$v_s(x)$ = Deformation corresponding to p_o, (m)

$w(x)$ = Nominal, unfactored dead load of the bridge superstructure and tributary substructure, (N/m)

(2) The computed factors, α, β and γ have units of (m^2), (N-m), and (N-m^2), respectively.

- Calculate the period of the bridge as:

$$T_m = 2\pi\sqrt{\frac{\gamma}{p_o g\alpha}} \qquad \text{(C4.3.2.2-4)}$$

where:

g = Acceleration of gravity, (m/sec^2)

- Using T_m and Equations 3.6.2-1, 3.6.2-4, or 3.64.2-5, calculate C_{sm}.
- Calculate the equivalent static earthquake loading $p_e(x)$ as:

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} w(x)v_s(x) \qquad \text{(C4.3.2.2-5)}$$

where:

C_{sm} = Dimensionless elastic seismic response coefficient given by Equations 3.6.2-1, 3.6.2-4, or 3.6.2-5

$p_e(x)$ = Intensity of the equivalent static seismic loading applied to represent the primary mode of vibration, (N/m)

- Apply loading $p_e(x)$ to the structure, and determine the resulting member force effects.

4.3.2.3 Uniform Load Method

The uniform load method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction of the base structure. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The elastic seismic response coefficient, C_{sm}, specified in Article 3.6.2 of these Specifications shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found.

Commentary C4.3.2.3

The uniform load method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100 percent. If such conservatism is undesirable, then the single-mode spectral analysis method specified in Article 4.3.2.2 of this Section is recommended. The procedure for the uniform load method is as follows:

- Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o as shown in Figure C4.3.2.2-1. The uniform loading p_o is applied over the length of the bridge; it has units of force per unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has units of length.

- Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions:

$$K = \frac{p_o L}{v_{sMAX}} \qquad \text{(C4.3.2.3-1)}$$

$$W = \int w(x)dx \qquad \text{(C4.3.2.3-2)}$$

where:

L = Total length of the bridge, (m)

v_{sMAX} = Maximum value of $v_s(x)$, (m)

$w(x)$ = Nominal, unfactored dead load of the bridge superstructure and tributary substructure, (N/m)

The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns, and footings. Other loads, such as live loads, may be included. Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios that are located in metropolitan areas where traffic congestion is likely to occur.

- Calculate the period of the bridge, T_m, using the expression:

$$T_m = 2\pi\sqrt{\frac{W}{gK}} \quad \text{(C4.3.2.3-3)}$$

where:

g = Acceleration of gravity (m/sec^2)

- Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_{sm}W}{L} \quad \text{(C4.3.2.3-4)}$$

where:

C_{sm} = Dimensionless elastic seismic response coefficient given by Equations 3.6.2-1, 3.6.2-4, or 3.6.2-5

p_e = Equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration (N/m)

- Calculate the displacements and member forces for use in design either by applying p_e, to the structure and performing a second static analysis or by scaling the results of the first step above by the ratio p_e/p_o.

4.3.3 Multimode Spectral Method

(1) The multimode spectral analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

(2) The number of modes included in the analysis should be at least three times the number of spans in the model. The design seismic response spectrum as specified in Article 3.6.1 of these Specifications shall be used for each mode.

(3) The member forces and displacements may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (*CQC*) method.

Commentary C4.3.3

(1) The multimode spectral method or the response spectrum method is used to obtain the maximum response occurring in a vibration system by the following procedure:

- Obtain the necessary number of vibration modes, natural periods and participation factors by eigenvalue analysis.
- Calculate the response (forces and displacements) corresponding to the frequency of each vibration mode using the design seismic acceleration response spectrum.
- Combine the respective response quantities of the vibration modes using the participation factors to obtain the maximum response of the vibration system.

(2) The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans. Higher number of modes may be used as necessary.

(3) Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems (Wilson et al., 1981).

If the *CQC* method is not readily available, alternative methods include the square root of the sum of the squares method (*SRSS*), but this method is best suited for combining responses from well-separated modes. For closely spaced modes, the absolute sum of the modal responses should be used.

4.3.4 Time-History Method

4.3.4.1 General

(1) Time history method shall be used as an alternative analysis method for critical bridges in Seismic Performance Zone 3 and 4 under Level 2 Earthquake Ground Motion. However, the DPWH may specify the use of time history analysis method for very important critical and irregular bridges.

(2) Any step-by-step time-history method of analysis used for either elastic or inelastic analysis shall satisfy the requirements of *Article 4.7 (Dynamic Analysis) of AASHTO LRFD Bridge Design Specifications (2012 or later)*.

(3) The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

(4) The time histories of input acceleration used to describe the earthquake loads shall be selected in accordance with Article 4.3.4.2 of this Section.

Commentary C4.3.4.1

Rigorous methods of analysis are required for critical structures, which are defined in Article 3.2 of these Specifications, and/or those that are geometrically complex or close to active earthquake faults. Time history methods of analysis are recommended for this purpose, provided care is taken with both the modeling of the structure and the selection of the input time histories of ground acceleration.

The time history response analysis method is a method used to obtain the response quantities of a vibration unit at discrete time intervals. In this method, the equations of motion of the vibration system is solved in sequence at small time increments using the earthquake ground motion time history as the external force. There are several methods used in the time history response analysis including the modal analysis, direct integration and the complex response method but the appropriate method shall be chosen based on the purpose of analysis during verification of seismic performance.

4.3.4.2 Acceleration Time Histories

(1) Developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.

(2) Response-spectrum-compatible time histories shall be used as developed from representative recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

(3) Where recorded time histories are used, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Each time history shall be modified to be response-spectrum compatible using the time-domain procedure.

(4) At least three response-spectrum-compatible time histories shall be used for each component of motion in representing the Level 2 EGM design earthquake (ground motions having seven percent (7%) probability of exceedance in 75 years). All three orthogonal components (x, y, and z) of design motion shall be input simultaneously when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.

(5) If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

(6) For near-field sites (D < 10 km), the recorded horizontal components of motion that are selected should represent a near-field condition and should be transformed into principal components before making them response-spectrum-compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

Commentary C4.3.4.2

(1) Characteristics of the seismic environment to be considered in selecting time histories include:

- Tectonic environment (e.g., subduction zone; shallow crustal faults),
- Earthquake magnitude,
- Type of faulting (e.g., strike-slip; reverse; normal),
- Seismic-source-to-site distance,
- Local site conditions, and
- Design or expected ground-motion characteristics (e.g., design response spectrum, duration of strong shaking, and special ground motion characteristics such as near-fault characteristics)

(2) Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps or PHIVOLCS data base, shall be used. Reference can also be made from deaggregation information on the USGS website: *http://geohazards.cr.usgs.gov*.

(3) It is desirable to select time histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time histories. Selection of time histories having similar earthquake magnitudes and distances, within reasonable ranges, are

especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground-motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 10 kms of an active fault, then intermediate-to-long-period ground-motion pulses that are characteristic of near-source time histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high short-period spectral content of near- source vertical ground motions should he considered.

(4) Ground motion modeling methods of strong motion seismology are being increasingly used to supplement the recorded ground motion database. These methods are especially useful for seismic settings for which relatively few actual strong motion recordings are available, such as in the central and eastern United States. Through analytical simulation of the earthquake rupture and wave propagation process, these methods can produce seismologically reasonable time series.

(5) Response spectrum matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time domain character of the recorded or simulated time histories. To minimize changes to the time domain characteristics, it is desirable that the overall shape of the spectrum of the recorded time history not be greatly different from the shape of the design response spectrum and that the time history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

(6) Where three-component sets of time histories are developed by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches for dealing with this scaling issue include:

- use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two,
- use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components, and
- Compromising on the scaling by using different factors as required for different components of a time-history set.

(7) While the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the higher horizontal component in each principal horizontal direction.

(8) The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

(9) Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee (CSABAC) on Soil-Foundation-Structure Interaction (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between

bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

4.4 MATHEMATICAL MODEL

4.4.1 General

(1) Mathematical model shall include loads, geometry, and material behavior of the structure, and, where appropriate, response characteristics of foundation. The choice of the model shall be based on the limit states investigated, the force effects being quantified, and the accuracy required.

(2) *Bridge Mathematical Model*

The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia of the structure. Each joint or node should have six degrees of freedom, three translational and three rotational. The structural mass should be lumped with a minimum of three translational inertia terms, consistent with Article 4.5 of this Section.

The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings.

The effects of live load shall be included as indicated in Article 1.5 of these Specifications.

(3) *Superstructure*

The superstructure should, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to joints at the ends of each span. Discontinuities should be included in the superstructure at the expansion joints and abutments. Care should be taken to distribute properly the lumped mass inertia effects at these locations.

The effects of earthquake restrainers or fall-down prevention devices at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.

Bearing supports should be modeled according to the type, function and behavior/response of the bearings.

Other devices attached to the bridge shall be modeled realistically to simulate the intended bridge behavior and displacements.

A more refined analytical model may be used for superstructure, when necessary, depending on the type and geometric configuration.

(4) *Substructure*

The intermediate columns or piers should also be modeled as space frame members. Generally, for short, stiff columns having lengths less than one-third of either the adjacent span lengths, intermediate nodes are not necessary. Long, flexible columns should be modeled with intermediate nodes at the third points in addition to the joints at the ends of the columns. The model should consider the eccentricity of the columns with respect to the superstructure.

Appropriate representation of the soil and /or rock that supports the bridge shall be included in the mathematical model of the foundation. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear spring constants derived from the coefficient of subgrade reactions given in Article 4.4.2 (for design) and Article 4.4.3 (for analysis) of this Section.

On bridge sites where the ground is prone to liquefaction, gross soil movement and liquefaction effects shall be considered in the analysis, as specified in Section 6 of these Specifications.

(5) Boundary Conditions

The boundary conditions shall represent the actual characteristics of support and continuity.

The foundation conditions shall be modeled in such a manner to represent the soil properties underlying the bridge, the soil-pile interaction, and the elastic properties of piles.

As an initial analysis model to determine the design forces for foundation, the piers shall be assumed to be fixed at the ground surface for seismic design.

Commentary C4.4.1

(1) The mathematical model of the bridge should accurately represent the physical and dynamic characteristics of the bridge and should include model elements for bearings, restrainers, support springs, devices for limiting displacements, etc. Stiffness properties of the materials in the mathematical model should be consistent with the anticipated behavior, as to either elastic or inelastic behavior.

(2) Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur.

(3) The need for a more refined modeling of foundations is a function of sensitivity of the structure to foundation movements. In some cases, the foundation model may be as simple as unyielding supports. In other cases, an estimate of settlement may be acceptable. Where the structural response is particularly sensitive to the boundary conditions, such as a fixed-end arch or in computing natural frequencies, rigorous modeling of the foundation should be made to account for the conditions present. In lieu of the rigorous modeling the boundary conditions may be varied to extreme bounds, such as fixed or free restraint, and envelopes of force effects considered.

4.4.2 Coefficients of Subgrade Reaction and Foundation Spring Constants for Design Use

(1) Although various elastic and inelastic models are used to represent foundation conditions using soil spring constants, this article refers to the method recommended by the *Japan Road Association (JRA)* to determine the coefficients of subgrade reaction and the subsequent corresponding soil spring constants to model the foundation for design use. Other acceptable methods to determine the coefficients of subgrade reaction and soil spring constants may be used, as approved by the DPWH.

The spring constants derived from this article shall be used to calculate the design forces in the foundations under static analysis. Spring constants for dynamic analysis under earthquake loading shall be that provided in Article 4.4.3 of this Section.

(2) The basic principle to determine the coefficient of subgrade reaction for the design of foundations is defined by Equation 4.4.2-1, as follows:

$$k = \frac{p}{\delta} \quad \text{..} \quad (4.4.2\text{-}1)$$

where:

k = Coefficient of subgrade reaction, (kN/m^3)

p = Subgrade reaction per unit area, (kN/m^2)

δ = Displacement, (m)

(3) The coefficient of subgrade reaction shall be determined, in principle, by using the modulus of deformation obtained from a variety of surveys and tests by considering the influence of loading width of foundations and other relevant factors.

Commentary C4.4.2

(1) This Specification utilizes the method recommended by the *Japan Road Association (JRA)* to model the foundation conditions and represent the soil properties underlying the bridge. The commentary provides a means to determine both the coefficients of vertical and horizontal subgrade reactions as detailed in this commentary.

(2) Since the soil layer is not an elastic material and has densities and compaction which vary with depth, the subgrade reaction-to-displacement curve indicates a nonlinear shape as shown in Figure C.4.4.2-1, even if the soil layer does not show a clear failure. Although the coefficient of subgrade reaction varies with displacement, this article gives the ratio between the subgrade reaction per unit area and the corresponding displacement as a coefficient of subgrade reaction.

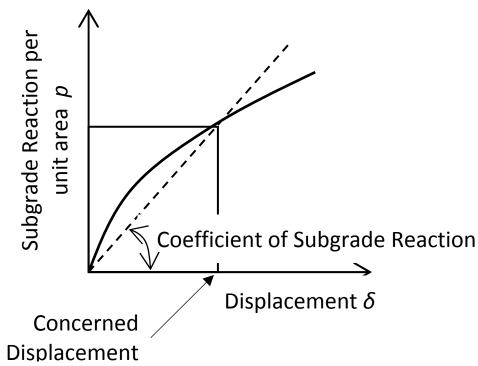


Figure C4.4.2-1 Coefficient of Subgrade Reaction

The coefficient of subgrade reaction is derived as a function of the modulus of deformation of the ground, as described in this Commentary. Since the modulus of deformation of the ground is a physical value that depends on the magnitude of strain occurring in the ground due to the load from the foundation, the pressure in the ground and loading duration, the value of the coefficient of subgrade reaction varies with these factors. In addition, the coefficient of subgrade reaction is influenced by the variations of the physical properties of the ground along the depth and the complexity of behavior of the loaded conditions of the structure in the actual state and under laboratory test.

The coefficient of subgrade reaction stipulated in this article is defined assuming a state in which the foundation exerts a static load on the ground. It is used in the design methods for foundations under ordinary conditions and in the static design methods for foundations during an earthquake. However, the coefficient of subgrade reaction and spring constants used for the analysis model of foundation (natural period calculation), foundation design and dynamic analysis should be separately calculated according to the requirements of Article 4.4.3 of this Section.

(3) Since the coefficient of subgrade reaction is one of the basic design constants necessary for finding the displacement and the reaction of the foundation, it must be carefully determined from the results of the surveys and tests.

A method to estimate the coefficient of subgrade reaction is described below. Since this is one of the several methods to estimate the subgrade reaction, the values obtained by means of the method mentioned below shall take into considerations the subgrade conditions, foundation design requirements, and other relevant factors. Other methods approved by the DPWH to obtain the coefficient of subgrade reaction may be used.

1) <u>Coefficient of Vertical Subgrade Reaction</u>

In general, the coefficient of vertical subgrade reaction shall be obtained by using Equation C.4.4.2-1.

$$k_V = k_{VO}\left(\frac{B_V}{0.3}\right)^{-3/4} \quad \text{.......................} \quad (C.4.4.2\text{-}1)$$

where:

k_v = Coefficient of vertical subgrade reaction, (kN/m^3)

k_{v0} = Coefficient of vertical subgrade reaction equivalent to the value of plate bearing test using a rigid disc of diameter 0.3m, and is obtained by using the following equation (if estimating it from the modulus of deformation found as a result of soil quality tests and surveys), (kN/m^3)

$$k_{VO} = \left(\frac{1}{0.3}\right)\alpha E_o \quad \text{.......................} \quad (C.4.4.2\text{-}2)$$

B_V = Equivalent loading width of foundation to be obtained by means of the following equation, (m). However, this should be substituted with the diameter, if the foundation has a circular base/bottom.

$$B_v = \sqrt{A_v} \quad \text{.......................} \quad (C.4.4.2\text{-}3)$$

E_O = Modulus of deformation of a soil layer at the point in issue, measured or estimated as shown in Table C. 4.4.2-1, (kN/m^2)

α = Coefficient to be used for estimating a coefficient of subgrade reaction, as shown in Table C. 4.4.2-1

A_V = Loading area in the vertical direction, (m^2)

2) *Coefficient of Horizontal Subgrade Reaction*

The coefficient of horizontal subgrade reaction should be obtained by using Equation C.4.4.2-4.

$$k_H = k_{HO}\left(\frac{B_H}{0.3}\right) \qquad \text{(C.4.4.2-4)}$$

where:

k_H = Coefficient of horizontal subgrade reaction, (kN/m^3)

k_{HO} = Coefficient of horizontal subgrade reaction corresponding to the value obtained by the plate bearing test using a rigid disc of diameter 0.3 m, (kN/m^3). This may also be determined from Equation C.4.4.2-5 in terms of the modulus of deformation obtained by various soil tests or field surveys:

$$k_{HO} = \left(\frac{1}{0.3}\right) aE_O \qquad \text{(C.4.4.2-5)}$$

B_H = Equivalent loading width of foundation to be obtained from Table C. 4.4.2-2, (m)

E_O = Modulus of deformation at the design location, measured or estimated by the procedures in Table C. 4.4.2-1, (kN/m^2)

α = Coefficient to be used for estimation of the coefficient of subgrade reaction, as shown in Table C. 4.4.2-1

A_H = Loading area of foundation perpendicular to the load direction, (m^2)

D = Loading width of foundation perpendicular to the load direction, (m)

B_e = Effective loading width of foundation perpendicular to the load direction, (m)

L_e = Effective embedment depth of a foundation, (m)

$1/\beta$ = Ground depth relating to the horizontal resistance and equal to or less than the effective embedment depth, (m)

β = Characteristic value of foundation, $\beta = \sqrt[4]{\frac{k_H D}{4EI}}$, (m^{-1})

EI = Flexural stiffness of foundation (kN·m^2)

Table C. 4.4.2-1 Modulus of Deformation E_O and α

Modulus of deformation E_O to be obtained by means of the following testing methods	α	
	Service and Strength Limit States (Ordinary Conditions)	Extreme Event Limit State (During Earthquake)
A value equal to ½ of the modulus of deformation to be obtained from a repetitive curve of a plate bearing test	1	2

Modulus of deformation E_O to be obtained by means of the following testing methods	α	
	Service and Strength Limit States (Ordinary Conditions)	Extreme Event Limit State (During Earthquake)
using a rigid disc of 30 cm in diameter		
Modulus of deformation to be measured in the bore hole	4	8
Modulus of deformation to be obtained by means of an unconfined or triaxial compression test of samples	4	8
Modulus of deformation to be estimated from E_O = 2,800 N using the N-value of the standard penetration test	1	2

Equation C.4.4.2-4 is used to evaluate the coefficient of horizontal subgrade reaction when treating the ground resistance in a linear manner and is calculated for the reference displacement. In JRA, the reference displacement at the linear range is recommended to be one percent (1%) of the foundation width (≤50 mm), which is taken as the allowable displacement required from the substructure. However, under earthquake loading this value is taken as a reference and may not necessarily be adhered to and may reach as much as 5% of the foundation width.

In general, the results from different tests for the same ground would vary according to the loading methods and dimensions employed. Thus, the coefficient of subgrade reaction should be obtained by modifying it according to the estimation procedures of E_o, as shown in Table C. 4.4.2-1. This table is derived by reviewing the correlations between different survey methods. The coefficient of subgrade reaction should be determined from the most current data, in consideration of the characteristics of the survey to obtain the modulus of deformation, and the relative properties of the foundation ground, rather than simply taking the average modulus of deformation obtained from the results of various tests.

Table C. 4.4.2-2 Equivalent Loading Width of Foundation B_H

Foundation Type	B_H	Remarks
Spread Foundation	$\sqrt{A_H}$	
Caisson Foundation	$B_e\left(\leq \sqrt{B_e L_e}\right)$	
File Foundation	$\sqrt{D/\beta}$	
Steel Pipe Sheet Pile Foundation	$\sqrt{D/\beta}\left(\leq \sqrt{B_e L_e}\right)$	Service and Strength Limit States and Level 1 Earthquake Ground Motion
	$B_e\left(\leq \sqrt{B_e L_e}\right)$	Level 2 Earthquake Ground Motion
Diaphragm (Slurry) Wall Foundation	$B_e\left(\leq \sqrt{B_e L_e}\right)$	

When analyzing the ground resistance of a pile foundation or steel pile sheet pile foundation as a linear spring, the equivalent loading width B_H should take a value of $\sqrt{D/\beta}$. In calculating B_H, the value of k_H during the normal condition may be taken as the average value from the design ground surface to the depth equal to $1/\beta$. Moreover, when the coefficients of horizontal subgrade reaction are calculated by assuming the ground as multiple layers, the equivalent loading width of each layer should take the value of B_H shown in Table C. 4.4.2-2.

For caisson and diaphragm (slurry) wall foundations whose analytical models are established by assuming a bi-linear ground resistance, the equivalent loading width can be the foundation width.

3) *Coefficient of Horizontal Shear Subgrade Reaction*

The coefficient of horizontal shear subgrade reaction should be obtained by using Equation C.4.4.2-6.

$$k_S = \lambda k_V \qquad \text{(C.4.4.2-6)}$$

where:

k_S = Coefficient of horizontal shear subgrade reaction, (kN/m^3)

λ = Ratio of coefficient of horizontal shear subgrade reaction to coefficient of vertical subgrade reaction; this coefficient can take the value $\lambda = 1/3 \sim 1/4$

k_V = Coefficient of vertical subgrade reaction, (kN/m^3)

The value of λ has been empirically estimated as 1/2 ~ 1/5. However, the above values are assumed so that the calculated displacement will be larger than the actual values. In this way, a higher factor of safety is used in view of the analytical formulation for calculating displacement.

4.4.3 Coefficients of Dynamic Subgrade Reaction and Foundation Spring Constants to Determine the Natural Period of the Structure (Dynamic Spring Constants Applied to the Analysis Model)

(1) The natural periods of the bridge shall be appropriately determined considering the effects of structural member deformations and the foundations. When the ground is unstable during earthquake, the natural periods shall be obtained without reducing the geotechnical parameters specified in Section 6 of these Specifications.

(2) During calculation of the natural period, the foundation shall be modeled considering the coefficient of subgrade reaction based on the ground stiffness equivalent to the ground deformation during an earthquake. The foundation spring constants for natural period calculation and dynamic analysis model shall be determined based on the coefficients of subgrade reaction using the dynamic soil properties during an earthquake.

Commentary C4.4.3

(1) When calculating the natural periods, the deformation effects of the structural members and the foundations shall be considered. However, the stiffness of the superstructure and substructure, including foundations, can generally be calculated assuming that the whole foundation section is effective during the verification of seismic performance.

The coefficients of subgrade reaction, for verification of seismic performance and calculation of natural period shall be obtained based on the stiffness of the ground which is equivalent to the deformation of the ground during an earthquake. The values of the coefficient of dynamic subgrade reaction shall be obtained from Equations C.4.4.3-1 and C.4.4.3-2.

$$k_{H0} = \frac{1}{0.3} E_D \qquad \text{(C.4.4.3-1)}$$

$$k_{V0} = \frac{1}{0.3} E_D \qquad \text{(C.4.4.3-2)}$$

$$E_D = 2(1 + \nu_D) G_D \qquad \text{(C.4.4.3-3)}$$

$$G_D = \frac{\gamma_t}{g} V_{SD}^2 \qquad \text{(C.4.4.3-4)}$$

where:

k_{H0} = Reference value of the coefficient of subgrade reaction in the horizontal direction, (kN/m^3)

k_{V0} = Reference value of the coefficient of subgrade reaction in the vertical direction, (kN/m^3)

E_D = Dynamic modulus of deformation of the ground, (kN/m^2)

ν_D = Dynamic Poisson's ratio of the ground

G_D = Dynamic shear deformation modulus of deformation of the ground, (kN/m^2)

γ_t = Unit weight of the ground, (kN/m^3)

g = Gravitational acceleration, (= 9.8 m/sec^2)

V_{SD} = Shear elastic wave velocity of the ground, (m/s)

V_{SD} for the ***i***-th soil layer can be calculated by Equation C.4.4.3-5 based on the V_{si} obtained from Equation C3.5.1-1.

$$V_{SDi} = c_V V_{si}\ , \quad c_V = \begin{cases} 0.8 (V_{Si} < 300\, m/s) \\ 1.0 (V_{Si} \geq 300\, m/s) \end{cases} \qquad \text{(C.4.4.3-5)}$$

where:

V_{SDi} = Average shear elastic wave velocity of the i-th soil layer to be used for calculation of spring constant representing resistance of foundation, (m/s)

V_{si} = Average shear elastic wave velocity of the i-th soil layer specified by Equation C3.5.1-1, (m/s)

c_V = Modification factor based on the degree of ground strain

However, when the shear elastic wave velocity V_{si} measured at the construction site is available, use of the measured value rather than the V_{si} obtained by Equation C3.5.1-1 is recommended.

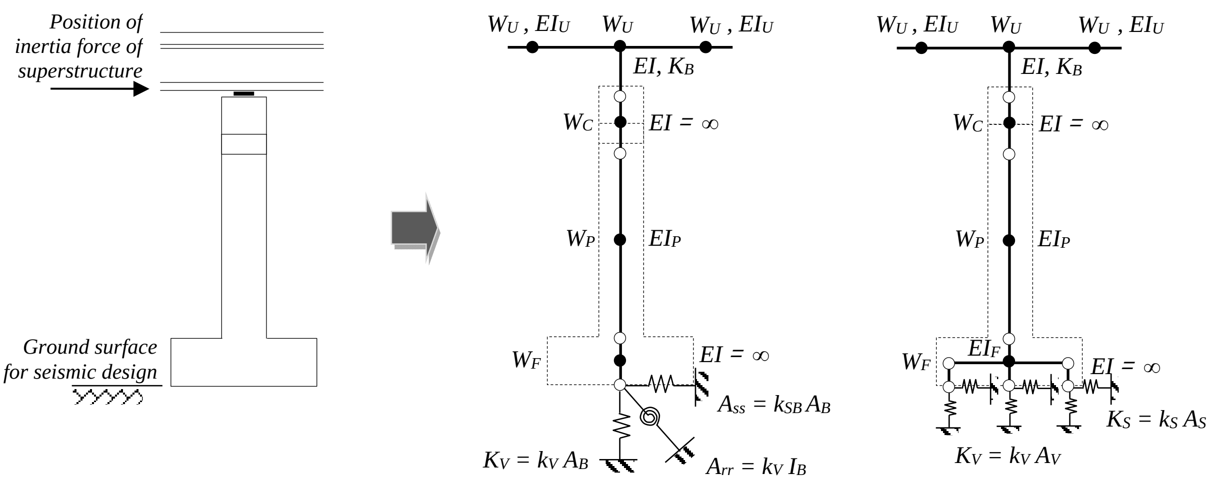
The dynamic Poisson's ratio of the ground can be set as:

Table C4.4.3-1 Dynamic Poisson's Ratio

Condition	ν_D (Dynamic Poisson's Ratio)
Above ground water level in alluvial or diluvial ground	0.45
Below ground water level in alluvial or diluvial ground	0.5
In soft rock	0.4
In hard rock	0.3

Reduction of the geotechnical parameter is not applied when calculating the natural period when an extremely soft clayey layer exists or a soil layer for which liquefaction affecting the bridge is expected in seismic design according to the provisions of Section 6.2 of these Specifications. This is because there are still many unclear points about the mechanism of excessive vibration characteristics caused by instability of the ground. When the natural period is obtained by reducing the geotechnical parameter under an unstable condition of the ground during an earthquake, the design seismic force is likely to be underestimated. Therefore, it is considered that a conservative value of the design seismic force shall be obtained.

(2) When determining the natural period for the design vibration unit, the distribution of stiffness and weights of the superstructure and substructure shall be modeled appropriately in accordance with the provisions of Article 4.5 of this Section. Although more sophisticated models may be used to discretize the bridge structure and its foundation, simplified models for substructure and foundation is indicated here for reference. Figures C4.4.3-1 illustrates the substructure model for spread foundation while Figure C4.4.3-2 illustrates the substructure model for pile foundation.



(a) Bridge Pier

(b) Footing Model using Lumped Springs at the End Support Node

(c) Footing Model Using Discrete Elements and Springs at Nodal Points

● : *Nodal point to model structure weight*	W_C : *Weight of coping beam at nodes*
○ : *Nodal point for section property change*	W_P : *Weight of pier body at nodes*
EI_U : *Superstructure flexural rigidity*	W_F : *Weight footing at nodes*
EI_P: *Substructure/pier flexural rigidity*	W_U : *Weight of superstructure at nodes*
EI_F : *Footing flexural rigidity*	K_B : *Spring constant of elastic bearing*

Figure C4.4.3-1 Natural Period Calculation and Dynamic Analysis Model for Spread Footing

1) *Spring Constants for Spread Footing Foundation*

When determining the natural periods of the bridge structure, the spread footing foundation may be modeled using a series of vertical, horizontal shear and rotational springs. Analytical models based on accepted and documented methods may be used in the analysis, as approved by DPWH. In these Specifications, either of the following model is recommended for analysis when determining the natural vibration periods under seismic load:

(a) The footing is modeled as vertical elements with vertical, horizontal shear, and rotational springs lumped at the end node support as shown in Figure C4.4.3-1(b). This model is recommended to simplify the dynamic analysis model.

(b) The footing is modeled using discrete elements representing the footing stiffness and material properties with vertical and horizontal shear springs used in the nodal supports as shown in Figure C4.4.3-1(c).

The spring constants to represent the resistance of the foundation for spread footing shall be calculated from Equation C.4.4.3-6 and C.4.4.3-7 as:

$$\left.\begin{aligned} A_{ss} &= k_S A_B \\ A_{sr} &= A_{rs} = 0 \\ A_{rr} &= k_V I_B \end{aligned}\right\} \quad \text{(C.4.4.3-6)}$$

$$K_V = k_V A_B \quad \text{(C.4.4.3-7)}$$

where:

A_{ss} = Horizontal spring constant due to foundation horizontal translation, (kN/m)

A_{sr} = Spring constant in the horizontal direction due to foundation rotation, (kN/rad)

A_{rs} = Spring constant in the rotational direction due to foundation horizontal translation, (kN-m/m)

A_{rr} = Rotational spring constant due to foundation rotation, (kN-m/rad)

k_S = Coefficient of shear subgrade reaction in the horizontal direction at the bottom of foundation derived from Equation C.4.4.2-6, (kN/m^3)

k_V = Coefficient of dynamic subgrade reaction in the vertical direction at the bottom of foundation derived from Equation C.4.4.2-1 and using k_{V0} values from Equation C.4.4.3-2, (kN/m^3)

I_B = Second moment of the bottom surface section of foundation, (m^4)

K_V = Axial spring constant of pile, (kN/m)

K_S = Horizontal spring constant of pile, (kN/m)

A_B = Area of bottom surface of foundation, (m^2)

A_V = Effective area of bottom surface of foundation corresponding to spring K_V, (m^2)

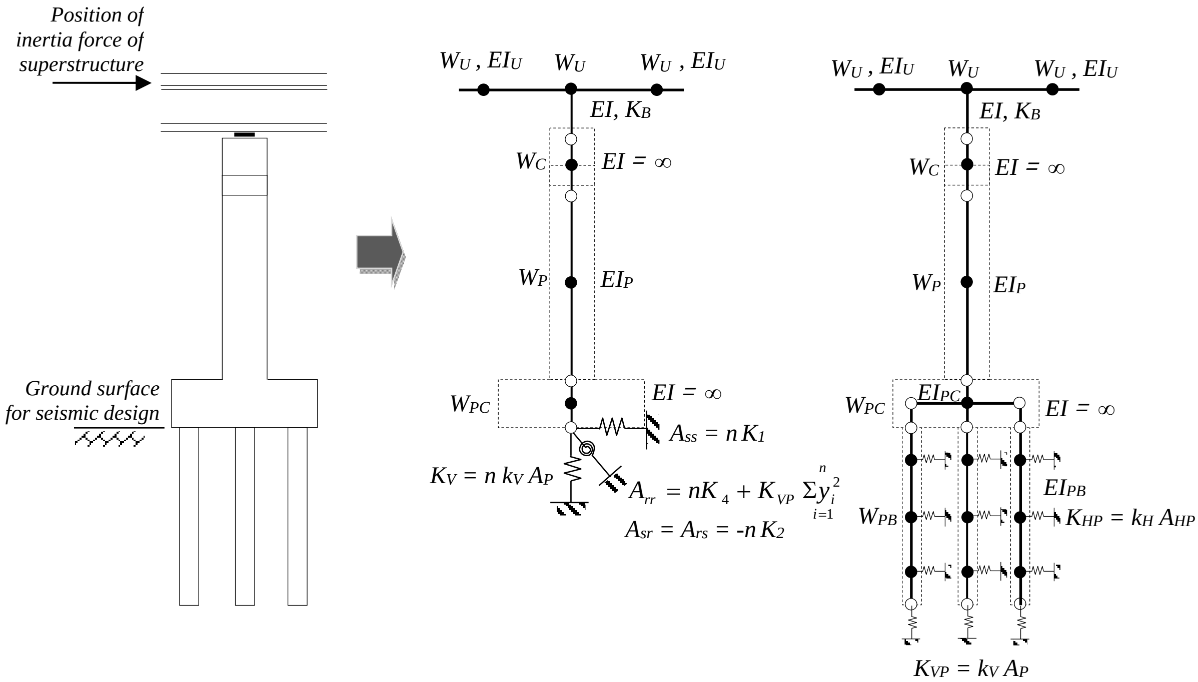
A_S = Effective area of bottom surface of foundation corresponding to spring K_S, (m^2)

2) *Spring Constants for Pile Foundation*

Similarly, in calculating the natural periods of the bridge structure supported by vertical piles symmetrical along the structure axis, the foundation may be modeled using a series of vertical, horizontal and rotational springs. Analytical models based on accepted and documented methods may also be used in the analysis, as approved by DPWH. In this Specification, either of the following model is recommended for analysis when determining the natural vibration periods under seismic load:

(a) The pile cap is modeled as vertical elements with the piles represented by vertical, horizontal, and rotational springs lumped at the end node support, as shown in Figure C4.4.3-2(b). The pile system is represented by foundation spring constants with properties considering all the piles in the group. This model is recommended to simplify the dynamic analysis model.

(b) The pile foundation is modeled using discrete elements representing the pile cap and pile body with corresponding stiffness and material properties. Vertical and horizontal springs are used at the nodal points in the piles, as shown in Figure C4.4.3-2(c), to represent the ground resistance. This model is recommended when designing pile foundation using plastic hinging forces from columns.



(a) Bridge Pier

(b) Footing Model using Lumped Springs at the End Support Node

(c) Footing Model Using Discrete Elements and Spring at Nodal points

● : *Nodal point to model structure weight*	W_C : *Weight of coping beam at nodes*
○ : *Nodal point for section property change*	W_P : *Weight of pier body at nodes*
EI_U : *Superstructure flexural rigidity*	W_{PC} : *Weight pile cap at nodes*
EI : *Substructure flexural rigidity*	W_U : *Weight of superstructure at nodes*
EI_{PC} : *Pile cap flexural rigidity*	W_{PB} : *Weight of pile body at nodes*
EI_{PB} : *Pile body flexural rigidity*	K_B : *Spring constant of elastic bearing*

Figure C4.4.3-2 Natural Period Calculation and Dynamic Analysis Model for Pile Foundation

For pile foundation with only vertical piles arranged symmetrically, the spring constants to represent the resistance of the foundation shall be calculated from Equation C.4.4.3-8 to C.4.4.3-10 as:

$$\left.\begin{aligned} A_{ss} &= nK_1 \\ A_{sr} &= A_{rs} = -nK_2 \\ A_{rr} &= nK_4 + K_{VP}\sum_{i=1}^{n} y_i^2 \end{aligned}\right\} \qquad \text{(C.4.4.3-8)}$$

$$K_{HP} = k_H A_{HP} \qquad \text{(C.4.4.3-9)}$$

$$K_{VP} = k_V A_P \qquad \text{(C.4.4.3-10)}$$

$$K_V = n K_{VP} \qquad \text{(C.4.4.3-11)}$$

where:

A_{ss} = Horizontal spring constant due to foundation horizontal translation, (kN/m)

A_{sr} = Spring constant in the horizontal direction due to foundation rotation, (kN/rad)

A_{rs} = Spring constant in the rotational direction due to foundation horizontal translation, (kN-m/m)

A_{rr} = Rotational spring constant due to foundation rotation, (kN-m/rad)

K_{HP} = Horizontal spring constant of pile section corresponding to area A_{HP}, (kN/m)

K_{VP} = Axial spring constant of a single pile in the direction of the pile axis, (kN/m)

K_V = Total axial spring constant of the pile group, (kN/m)

k_V = Coefficient of dynamic subgrade reaction in the vertical direction at the bottom of pile derived from Equation C.4.4.2-1 and using k_{V0} values from Equation C.4.4.3-2, (kN/m^3)

k_H = Coefficient of dynamic subgrade reaction in the horizontal direction at the pile section corresponding to area A_{HP} derived from Equation C.4.4.2-4 and using k_{H0} values from Equation C.4.4.3-1, (kN/m^3)

n = Total number of piles

y_i = y-coordinate of the pile head of i-th pile, (m)

K_1, K_2, K_3, K_4 = Radial spring constants of the piles in the perpendicular direction to the pile axis at the pile head as given in Table C4.4.4.3-2, (kN/m, kN/rad, kN-m/m, kN-m/rad)

A_{HP} = Effective projected vertical area of the ground corresponding to pile spring K_{HP}, (m^2)

A_P = Area of bottom surface of each pile, (m^2)

3) Radial Spring Constants of Piles

The radial spring constant of a single pile is evaluated on the basis of the beam theory for elastic foundations using the coefficient of horizontal subgrade reaction.

The radial spring constants K_1 to K_4 of a pile are thus defined below:

K_1, K_3 : radial force and bending moment (kN-m/m) to be applied on a pile head when displacing the head by a unit displacement in the radial direction while keeping it from rotating, (kN/m)

K_2, K_4 : radial force and bending moment (kN-m/rad) to be applied on a pile head when rotating the head by a unit rotation in the radial direction while keeping it from moving in a radial direction, (kN/rad)

These spring constants can be obtained from the relationship between the load and displacement to be computed based on the theory of a beam on an elastic foundation using the coefficient of horizontal subgrade reaction.

3a) Piles with a semi-infinite length ($\beta L_e \geq 3$)

If the coefficient of horizontal subgrade reaction is constant irrespective of the depths and if the embedded depth of a pile is sufficiently long, the constants can be computed from Table C4.4.3-2 by using the Hayashi-Chang Theory.

Table C4.4.3-2 Radial Spring Constants of a Pile

	Rigid frame of pile head		Hinged frame of pile head	
	$h \neq 0$	$h = 0$	$h \neq 0$	$h = 0$
K_1	$\dfrac{12EI\beta^3}{(1+\beta h)^3+2}$	$4EI\beta^3$	$\dfrac{3EI\beta^3}{(1+\beta h)^3+0.5}$	$2EI\beta^3$
K_2, K_3	$K_1\dfrac{\lambda}{2}$	$2EI\beta^2$	0	0
K_4	$\dfrac{4EI\beta}{(1+\beta h)}\bullet\dfrac{(1+\beta h)^3+0.5}{(1+\beta h)^3+2}$	$2EI\beta$	0	0

where:

β : characteristic value of a pile, $\beta = \sqrt[4]{\dfrac{k_H D}{4EI}}\left(m^{-1}\right)$

λ : $h + \dfrac{1}{\beta}(m)$

k_H : Coefficient of horizontal subgrade reaction determined from Equation C.4.4.2-4, (kN/m^3). When calculating the natural period for the bridge structure, k_{H0} is determined from Equation C.4.4.3-1, however, for design of pile foundation k_{H0} is determined from Equation C.4.4.2-5.

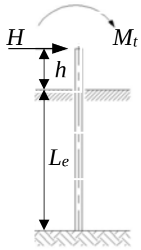
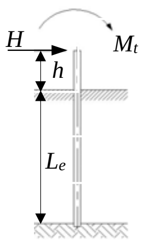
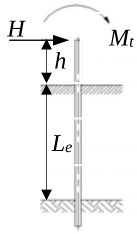
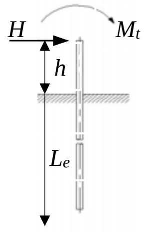
D : Pile diameter, (m)

For a steel pipe soil cement pile, use the diameter of the soil cement column.

EI : Bending rigidity of the pile, (kN-m^2).

For the case of the steel pipe soil cement pile rigidity, the rigidity of the steel pipe alone may be taken because the soil cement contributes little to the rigidity if its unconfined compressive strength is about 1 N/mm^2.

h : axial length of the pile above design ground surface, (m).

Conditions of Pile Tip	Piles of finite lengths (1<βLe<3)			Piles of semi-infinite lengths ($\beta L_e \geq 3$)
	Free (f)	Hinged (h)	Fixed (c)	
Schematic Drawing				
Spring Constants	$K_1ø_1^f$ $K_2ø_2^f$ $K_3ø_3^f$ $K_4ø_4^f$	$K_1ø_1^h$ $K_2ø_2^h$ $K_3ø_3^h$ $K_4ø_4^h$	$K_1ø_1^c$ $K_2ø_2^c$ $K_3ø_3^c$ $K_4ø_4^c$	K_1 K_2 K_3 K_4

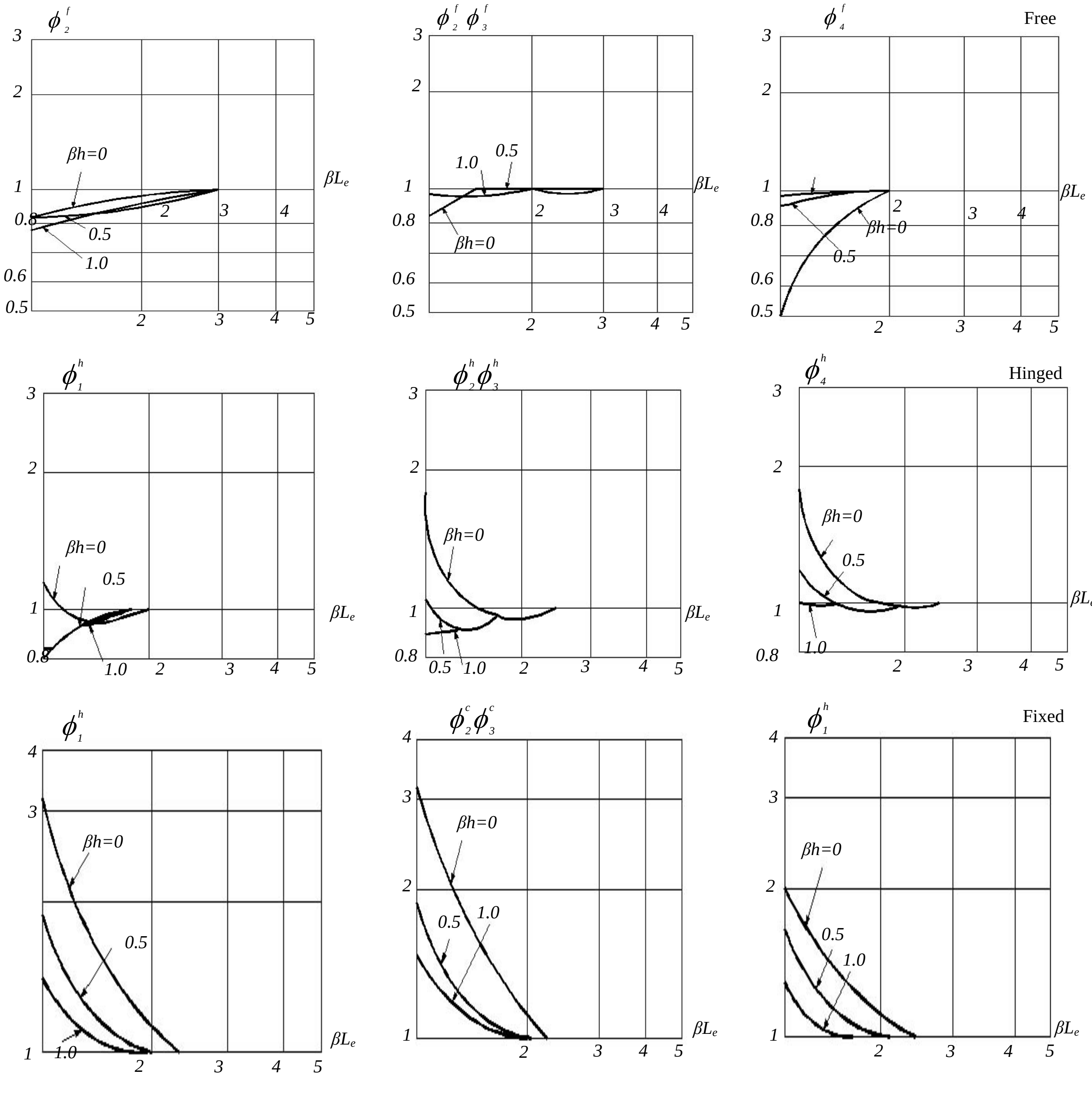


Figure C4.4.3-3 Corrective Coefficient of Radial Spring Constants of Piles of Finite Lengths

3b) Piles with a finite length ($1<\beta L_e<3$)

When determining the radial displacement and sectional force for a pile with a finite length, the bearing condition of the pile tip need to be considered since it affects these values. However, if the pile tip is embedded into a good quality supporting layer to a depth similar to the pile diameter, the pile can be generally regarded as hinged (pile tip).

When it is possible to assume that a coefficient of horizontal subgrade reaction (k_H) is constant in the direction of depth, the displacement method can be calculated by using values $K_1\phi_1$, $K_2\phi_2$, $K_3\phi_3$ and $K_4\phi_4$ which are obtained by multiplying the radial spring constants K_1, K_2, K_3 and K_4 by the corrective coefficient ϕ_i.

This corrective coefficient ϕ_i is a function of βL_e and βh and have values shown in Figure C4.4.3-3. However, Figure C4.4.3-3 should be applied within a range of $1<\beta L_e<3$.

4.5 DYNAMIC ANALYSIS REQUIREMENTS

4.5.1 Basic Requirements

(1) The stiffness, mass, and damping characteristics of the structural components shall be modeled to analyze the dynamic behavior of bridges.

(2) The minimum number of degrees-of-freedom included in the analysis shall be based upon the number of natural frequencies to be obtained and the reliability of the assumed mode shapes. The model shall be compatible with the accuracy of the solution method. Dynamic models shall include relevant aspects of the structure and the excitation. The relevant aspects of the excitation may include:

- Distribution of mass,
- Distribution of stiffness, and
- Damping characteristics.

The relevant aspects of excitation may include:

- Frequency of the forcing function,
- Duration of application, and
- Direction of application.

Commentary C4.5.1

The number of frequencies and mode shapes necessary to complete a dynamic analysis should be estimated in advanced or determined as an early step in a multistep approach. Having determined that number, the model should be developed to have a larger number of applicable degrees-of-freedom. Sufficient degrees-of-freedom should be included to represent the mode shapes relevant to the response sought. One rule-of-thumb is that there should be twice as many as degrees-of-freedom as required frequencies.

The number of degrees-of-freedom and the associated masses should be selected in a manner that approximates the actual distributive nature of mass. The number of required frequencies also depends on the frequency content of the forcing function.

4.5.2 Distribution of Masses

The modelling of mass shall be made with consideration of the degree of discretization in the model and the anticipated motions.

Commentary C4.5.2

The distribution of stiffness and mass should be modeled in a dynamic analysis. The discretization of the model should account for geometric and material variation in stiffness and mass.

The selection of the consistent lump mass formulation is a function of the system and the response sought and is difficult to generalize. For distributive mass systems modeled with polynomial shape functions in which the mass is associated with distributive stiffness such as beam, a consistent mass formulation is recommended (Paz, 1985). In lieu of a consistent formulation, lumped masses may be associated at the translational degrees-of-freedom, a manner that approximates the distributive nature of the mass (Clough and Penzian, 1975).

For systems with distributive mass associated with larger stiffness such as in-plane stiffness of a bridge deck, the mass may be properly modeled as lumped. The rotational inertia effects should be included where significant.

4.5.3 Stiffness

The bridge shall be modeled to be consistent with the degree-of-freedom chosen to represent the natural modes and frequencies of vibration. The stiffness of the elements of the model shall be defined to be consistent with the bridge being modeled.

Commentary C4.5.3

In seismic analysis, nonlinear effects of which decrease stiffness, such as inelastic deformation and cracking, should be considered.

Although the use of uncracked section properties are acceptable to model the structural components such as columns or walls in the dynamic analysis, AASHTO recommends that reinforced concrete columns and walls in Seismic Performance Zones 2, 3 and 4 be analyzed using cracked section properties where plastic hinging is anticipated. The use of cracked section properties for concrete columns where plastic hinging is anticipated results to longer period than that of members modeled with uncracked section properties. Where inelastic behavior or plastic hinging is anticipated, AASHTO recommends that a moment of inertia equal to one-half that of the uncracked section may be used in the plastic hinge zone. However, if the member is taken to behave linearly without plastic hinging, the full uncracked section property shall be applied.

4.5.4 Damping

Equivalent viscous damping may be used to represent energy dissipation.

Commentary C4.5.4

Damping may be neglected in the calculation of natural frequencies and associated nodal displacements. The effects of damping should be considered where a transient response is sought.

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values recommended in AASHTO may be used for the equivalent viscous damping ratio:

- Concrete construction : two percent (2%)
- Welded and bolted steel construction : one percent (1%)
- Timber : five percent (5%)

4.5.5 Natural Frequencies

All relevant damped modes and frequencies shall be considered.

4.5.6 Inelastic Dynamic Responses

4.5.6.1 General

During a major earthquake, energy may be dissipated by one or more of the following mechanisms:

- Elastic and inelastic deformation of the object that may collide with the structure,
- Inelastic deformation of the structure and its attachments,
- Permanent displacement of the masses of the structure and its attachments, and
- Inelastic deformation of special-purpose mechanical energy dissipators.

4.5.6.2 Plastic Hinges and Yield Lines

For the purpose of analysis, energy absorbed by inelastic deformation in a structural component may be assumed to be concentrated in plastic hinges and yield lines. The location of these sections may be established by successive approximation to obtain a lower bound solution for the energy absorbed. For these sections, moment-rotation hysteresis curves may be determined by using verified analytic material models.

4.6 MINIMUM SEAT LENGTH REQUIREMENTS

(1) Adequate measures against unseating of superstructures shall be taken when the superstructure separates structurally from the substructure, and with large relative displacements.

(2) Support lengths at expansion bearings without restrainers, shock transmission units (STUs) or dampers shall be designed to either accommodate the greater of the maximum calculated

displacement based on Article 4.3, except for bridges in Zone 1, or the empirical seating or support length, S_{EM}, specified in Section 7 (Unseating Prevention System). Section 7 specifies an unseating prevention system consisting of seating length at the girder support, unseating prevention device (or restrainers), device to limit excessive displacement and device to prevent the superstructure from settling.

Commentary C4.6

(1) The seating or support lengths are equal to the length of the overlap between the girder and the seat as discussed in Section 7. To satisfy the minimum values for S_{EM}, the overall seat width will be larger than S_{EM} by an amount equal to movements due to prestress shortening, creep, shrinkage, and thermal expansion/contraction. The minimum value for S_{EM} shall include an arbitrary allowance for cover concrete at the end of the girder and face of the seat. If the above cover is used at these locations, S_{EM} should be increased accordingly.

4.7 *P-Δ* REQUIREMENTS

(1) The displacement of any column or pier in the longitudinal or transversal direction shall satisfy:

$$\Delta P_u < 0.25\phi M_n \qquad (4.7\text{-}1)$$

in which:

$$\Delta = 12 R_d \Delta_e \qquad (4.7\text{-}2)$$

- If $T < 1.25T_s$, then:

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_S}{T} + \frac{1}{R} \qquad (4.7\text{-}3)$$

- If $T \geq 1.25T_s$, then:

$$R_d = 1 \qquad (4.7\text{-}4)$$

where:

Δ = Displacement of the point of contraflexure in the column or pier relative to the point of fixity for the foundation, (mm)

Δ_e = Displacement calculated from elastic seismic analysis, (mm)

T = Period of fundamental mode of vibration, (sec)

T_s = Corner period specified in Article 3.6.2, (sec)

R = Factor specified in Article 3.8

P_u = Axial load on column or pier, (N)

ϕ = Flexural resistance factor for column, ϕ = 0.90 for columns with either spiral or tie reinforcement) as specified in *Article 5.10.11.4.1b of AASHTO LRFD Bridge Design*

		Specifications, 2012
M_n	=	Nominal flexural strength of column or pier calculated at the axial load on the column or pier, (N-mm)

Commentary C4.7

(1) Bridges subject to earthquake ground motion may be susceptible to instability due to P-Δ effects. Inadequate strength can result in ratcheting of structural displacements to larger and larger values causing excessive ductility demand on plastic hinges in the columns, large residual deformations, and possibly collapse. The maximum value for Δ given in this Article is intended to limit the displacements such that P-Δ effects will not significantly affect the response of the bridge during an earthquake.

(2) P-Δ effects lead to a loss in strength once yielding occurs in the columns of a bridge. In severe cases, this can result in the force-displacement relationship having a negative slope once yield is fully developed. The value for Δ given by Equation 4.7-1 is such that this reduction in strength is limited to 25 percent of the yield strength of the pier or bent.

(3) An explicit P-Δ check was not required in the previous edition of the *AASHTO LRFD Bridge Design Specifications* but has been introduced herein because two conservative provisions have been relaxed in this revised edition. These are:

- The shape of the response spectrum (Figure 3.6.1-1) has been changed from being proportional to $1/T^{2/3}$ to $1/T$. The reason for the $1/T^{2/3}$ provision in the previous edition was to give conservative estimates of force and displacement in bridges with longer periods (>1.0 sec) which, in an indirect way, provided for such effects as P-Δ. With the change of the spectrum to being proportional to $1/T$, an explicit check for P-Δ is now required.
- The flexural resistance factor, ϕ, for seismic design of columns with high axial loads has been increased from a minimum value of 0.5 to 0.9 *Article 5.10.11.4.1b* of the *AASHTO LRFD Bridge Design Specifications (2012)*. Use of a low resistance factor led to additional strength being provided in heavily loaded columns that could be used to offset reductions due to P-Δ, in the previous edition. The increased value for ϕ now permitted in *Section 5 of the AASHTO LRFD Bridge Design Specifications (2012)* is a second reason for requiring an explicit check for P-Δ.

APPENDIX 4A:
OVERSTRENGTH RESISTANCE

COLUMN OVERSTRENGTH

This Appendix is taken from ***Appendix B*** *of the AASHTO LRFD Bridge Design Specification, 2012* and defines the forces resulting from plastic hinging (i.e., a column reaching its ultimate moment capacity) in the columns and presents two procedure:

- One is for a single column hinging about its two principal axes; this is also applicable for piers and bents acting as a single columns.
- The other procedure is for a multiple column bent in the plane of the bent.

The forces are based on the potential overstrength resistance that can occur in columns. The overstrength resistance results from actual properties being greater than the minimum specified values and implemented by specifying resistance factors greater than unity. This fact must be accounted for when forces generated by yielding of the column are used as design forces. Generally, overstrength resistance depends on the following factors:

1) The actual size of the column and the actual amount of reinforcing steel.
2) The effect of an increased steel strength over the specified f_y and for strain hardening effects.
3) The effect of an increased concrete strength over the specified f_c and confinement provided by the transverse steel. Also, with time, concrete will gradually increase in strength.
4) The effect of an actual concrete ultimate compressive strain above 0.003.

Commentary 4A

(A) Column Size and Reinforcement Configuration

The design engineer should select the minimum column section size and steel reinforcement ratio when satisfying structural design requirements. As these parameters increase, the overstrength increases. This may lead to an increase in the foundation size and cost. A size and reinforcement ratio which forces the design below the nose of the interaction curve is preferable, especially in high seismic areas. However, the selection of size and reinforcement must also satisfy architectural, and perhaps other requirements, which may govern the design.

(B) Increase in Reinforcement Strength

Almost all reinforcing bars will have a yield strength larger than the minimum specified value which may be up to 30 percent higher, with an average increase of 12 percent. Combining this increase with the effect of strain hardening, it is realistic to assume an increased yield strength of 1.25 f_y, when computing the column overstrength.

(C) Increase in Concrete Strength

Concrete strength is defined as the specified 28-day compression strength; this is a low estimate of the strength expected in the field. Typically, conservative concrete batch designs result in actual 28-day strengths of about 20-25 percent higher than specified. Concrete will also continue to gain strength with age. Tests on cores taken from older California bridges built in the 1950s and 1960s have consistently yielded compression strength in excess of 1.5 f'_c. Concrete compression strength is further enhanced by the possible confinement provided by the transverse reinforcement. Rapid loading due to seismic forces could also result in significant increase in strength, i.e., strain rate effect. In view of all the above, the actual concrete strength when a seismic event occurs is likely to significantly exceed the specified 28-day strength. Therefore, an increased concrete strength of 1.5 f'_c could be assumed in the calculation of the column overstrength resistance.

(D) Ultimate Compressive Strain (ε_χ)

Although test on unconfined concrete show 0.003 to be a reasonable strain at first crushing, tests on confined column sections show a marked increase in this value. The use of such a low extreme fiber strain is a very conservative estimate of strains at which crushing and spalling first develop in most columns, and considerably less than the expected strain at maximum response to the design seismic event. Research has supported strains on the order of 0.01 and higher as the likely magnitude of ultimate compressive strain. Therefore, designers could assume a value of ultimate strain equal to 0.01 as a realistic value.

For calculation purposes, the thickness of clear concrete cover used to compute the section overstrength shall not be taken to be greater than 2.0 in. This reduced section shall be adequate for all applied loads associated with the plastic hinge.

(E) Overstrength Capacity

The derivation of the column overstrength capacity is depicted in Figure 4A-1. The effect of higher material properties than specified is illustrated by comparing the actual overstrength curve, computed with realistic f'_c, f_y, and ε_c values, to the nominal strength interaction curve, P_n, M_n. It is generally satisfactory to approximate the overstrength capacity curve by multiplying the nominal moment strength by the 1.3 factor for axial loads below the nose of the interaction curve, i.e., P_n, 1.3 M_n curve. However, as shown, this curve may be in considerable error for axial loads above the nose of the interaction curve. Therefore, it is recommended that the approximate overstrength curve be obtained by multiplying both P_n, and M_n by ϕ = 1.3, i.e., 1.3 P_n, 1.3 M_n . This curve follows the general shape of the actual curve very closely at all levels of axial loads. In the light of the above discussion, it is recommended that:

1) For all bridges with axial loads below P_b, the overstrength moment capacity shall be assumed to be 1.3 times the nominal moment capacity.

2) For bridges in Seismic Performance Zone 3 and 4 with operational classification of “other”, and for all bridges in Seismic Performance Zone 2 for which plastic hinging has been invoked, the overstrength curve for axial loads greater than P_b shall be approximately by multiplying both P_n, and M_n by ϕ = 1.3.

3) For bridges in Seismic Performance Zone 3 and 4 with operational classification of “essential” or “critical”, the strength curve for axial loads greater than P_b shall be computed using realistic values for f'_c, f_y, and ε_c as recommended in Table 4A-1 or from values based on actual test

results. The column overstrength, thus calculated, should not be less than the value estimated by the approximate curve based on 1.3 P_n, 1.3 M_n.

Table 4A-1: Recommended Increased Values of Materials Properties

Increased f_y (minimum)	1.25 f_y
Increased f'_c	1.5 f'_c
Increased ε_c	0.01

(F) Shear Failure

The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges. For flared columns, these may occur at the top and bottom of the flare. For multiple column bents with a partial-height wall, the plastic hinges will probably occur at the top of the wall unless the wall is structurally separated from the column. For columns with deeply embedded foundations, the plastic hinge may occur above the foundation mat or pile cap. For pile bents, the plastic hinge may occur above the calculated point of fixity. Because of the consequences of a shear failure, it is recommended that conservatism be used in locating possible plastic hinges such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design.

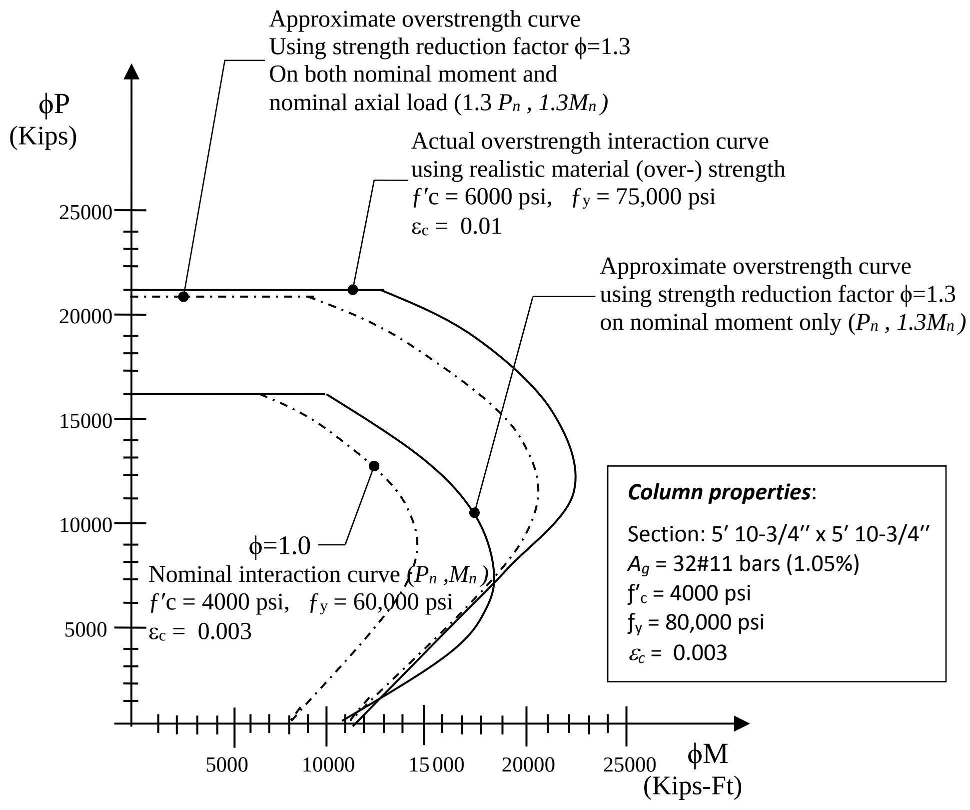


Figure 4A-1 Development of Approximate Overstrength Interaction Curves from Nominal Strength Curves after Gajer and Wagh (1994)

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SECTION 5:

DESIGN REQUIREMENTS

5.1 GENERAL

(1) This Section describes the design requirements for earthquake effects on bridges under the extreme event limit state. For other design and detail requirements not specified in this Section the design shall comply with the requirements of the *AASHTO LRFD Bridge Design Specifications (2012 or later)* corresponding to the components, materials and construction provisions. The design shall also comply with the requirements of the updated and revised DPWH Guide Specifications.

(2) The requirements for seismic force effects and the design forces for each seismic performance zone are given in this Specifications. The requirements for the design of foundation (spread foundation and pile foundation) are provided in this Section with the provisions for pile foundation design based on the Japan Road Association procedures. Alternative methods for foundation design may be used with the approval of DPWH.

Commentary C5.1

The provisions covered in this Section focuses on the design requirements for extreme event limit state under earthquake effects. The design of members not covered under this Section and the design for other limit states shall comply with the *AASHTO LRFD Bridge Design Specifications (2012 or later)*. However, since the DPWH is in the process of updating and revising the design guidelines and specifications, the requirements for design shall also be consistent with the updated and revised DPWH Design Guidelines.

5.2 COMBINATION OF SEISMIC FORCE EFFECTS

(1) The elastic seismic force effects on each of the principal axes of a member or component resulting from analyses in the two perpendicular directions shall be combined to form two load cases as follows:

- 100 percent of the absolute value of the force effects in one of the perpendicular directions combined with 30 percent of the absolute value of the force effects in the second perpendicular direction, and
- 100 percent of the absolute value of the force effects in the second perpendicular direction combined with 30 percent of the absolute value of the force effects in the first perpendicular direction.

(2) Where foundation and/or column connection forces are determined from plastic hinging of the columns specified in Article 5.3.4.3 of this Section, the resulting force effects may be determined without consideration of combined load cases specified herein. For the purpose of this provision, "column connection forces" shall be taken as the shear and moment, computed on the basis of plastic hinging. The axial load shall be taken as that resulting from the appropriate load combination with the axial load, if any, associated with plastic hinging taken as *EQ*. If a pier is designed as a column as specified in Article 3.8.2 of these Specifications, this exception shall be taken to apply for the weak direction of the pier where force effects resulting from plastic hinging are used; the combination load cases specified must be used for the strong direction of the pier.

Commentary C5.2

(1) The exception to these load combinations indicated at the end of this Section should also apply to bridges in Seismic Performance Zone 2 where foundation forces are determined from plastic hinging of the columns.

5.3 CALCULATION OF DESIGN FORCES

5.3.1 General

(1) Bridges shall be designed for strength and stability under Level 2 Earthquake Ground Motion (EGM). Design forces and displacements shall be that derived from analysis using Level 2 EGM.

(2) For single-span bridges, regardless of seismic performance zone, the minimum design connection force effect in the restrained direction between the superstructure and the substructure shall not be less than the product of the acceleration coefficient, A_S, specified in Equation 3.6.2-2, and the tributary permanent load.

(3) Minimum support lengths at expansion bearings of multi-span bridges shall either comply with Article 4.6 of these Specifications or STUs, and dampers shall be provided,

Commentary C5.3.1

(1) This Article refers to superstructure effects carried into the substructure. Abutments on multi-span bridges and retaining walls are subject to acceleration-augmented soil pressures as specified in *Articles 3.11.4 and 11.6.5 of the AASHTO LRFD Bridge Design Specifications (2012)*. Although abutments and wingwalls on single-span structures are not fully covered at this time, the Engineer should use judgment in this area. Reference shall be made to *Section 11 (Walls, Abutments and Piers)* of the *AASHTO LRFD Bridge Design Specifications 2012 (or later versions).*

5.3.2 Seismic Performance Zone 1

(1) For bridges in Seismic Performance Zone 1 where the acceleration coefficient, A_S, as specified in Equation 3.6.2-2, is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

(2) For all other sites in Seismic Performance Zone 1, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

(3) The horizontal design connection force shall be addressed from the point of application through the substructure and into the foundation elements.

(4) For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

(5) If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force

shall be the permanent load reaction at that bearing.

(6) Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in Seismic Performance Zone 1 and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Commentary C5.3.2

(1) These provisions arise because, as specified in Section 4 of these Specifications, seismic analysis for bridges in Seismic Performance Zone 1 is not generally required. These default values are used as minimum design forces in lieu of rigorous analysis. The division of Seismic Performance Zone 1 at a value for the acceleration coefficient, A_S, of 0.05 recognizes that, in parts of the country with very low seismicity, seismic forces on connections are very small. However, these Specifications sets the minimum *PGA* at 0.20 with S_{D1} values that corresponds to Seismic Performance Zones 3 or 4. The requirements for Seismic Performance Zones 1 and 2 are presented here for completeness.

(2) If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing without a device to limit excessive displacement, there are no restrained directions due to the flexibility of the bearings. However, the requirements for Type A bearing of Section 7 of these Specifications shall be complied with.

(3) Lateral connection forces are transferred from the superstructure into the foundation elements through the substructure. The force effects in this load path from seismic and other lateral loads should be addressed in the design. If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing, there may be no fully restrained directions due to the flexibility of the bearings. However, the forces transmitted through these bearings to substructure and foundation elements should be determined in accordance with this Article and *Article 14.6.3 of the AASHTO LRFD Bridge Design Specifications (2012)*.

(4) The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{EQ}, used in conjunction with Article 1.5 of these Specifications and Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications (2012)*.

5.3.3 Seismic Performance Zone 2

(1) Structures in Seismic Performance Zone 2 shall be analyzed according to the minimum requirements specified in Articles 4.1 to 4.3 of Section 4.

(2) Except for foundations, seismic design forces for all components, including pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Article 5.2 of this Section, by the appropriate response modification factor, *R*, specified in Table 3.8.1-1.

(3) Seismic design forces for foundations, other than pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Article 5.2 of this Section, by half of the response modification factor, *R*, from Table 3.8.1-1, for the substructure component to which it is attached. The value of *R/2* shall not be taken as less than 1.0.

(4) Where a group load other than Extreme Event I, as specified in *Table 3.4.1-1 (Load Combinations and Load Factors)* of the *AASHTO LRFD Bridge Design Specifications (2012)*, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than

those calculated using the procedure specified above, due to a possible overstrength of the columns, shall be considered.

Commentary C5.3.3

(1) This Article specifies the design forces for foundations which include the footings, pile caps and piles. The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges in Seismic Performance Zone 2. However, if seismic forces do not govern the design of columns and piers there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. For example, this may occur due to unintended column overstrengths which may exceed the capacity of the foundations. An estimate of this effect may be found by using a resistance factor, ϕ, of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. It is also possible that even in cases when seismic loads govern the column design, the columns may have insufficient shear strength to enable a ductile flexural mechanism to develop, but instead allow a brittle shear failure to occur. Again, this situation is due to potential overstrength in the flexural capacity of columns and could possibly be prevented by arbitrarily increasing the column design shear by the overstrength factor cited above.

(2) Conservatism in the design, and in some cases underdesign, of foundations and columns in Seismic Performance Zone 2 based on the simplified procedure of this Article has been widely debated (Gajer and Wagh, 1994). In light of the above discussion, it is recommended that for critical or essential bridges in Seismic Performance Zone 2 consideration should be given to the use of the forces specified in Article 5.3.4.3f for foundations in Seismic Performance Zone 3 and Zone 4. Ultimate soil and pile strengths are to be used with the specified foundation seismic design forces.

5.3.4 Seismic Performance Zones 3 and 4

5.3.4.1 General

(1) Structures in Seismic Performance Zones 3 and 4 shall be analyzed according to the minimum requirements specified in Articles 4.1 and 4.3 of these Specifications.

(2) The design forces of each component shall be taken as the lesser of those determined using:

- the provisions of Article 5.3.4.2 of this Section; or
- the provisions of Article 5.3.4.3 of this Section,

for all components of a column, column bent and its foundation and connections.

Commentary C5.3.4.1

(1) In general, the design forces resulting from an *R*-factor and inelastic hinging analysis will be less than those from an elastic analysis. However, in the case of architecturally oversized column(s), the forces from an inelastic hinging analysis may exceed the elastic forces in which case the elastic forces may be used for that column, column bent and its connections and foundations.

5.3.4.2 Modified Design Forces

Modified design forces shall be determined as specified in Article 5.3.3 of this Section, except that for foundations the *R*-factor shall be taken as 1.0.

Commentary C5.3.4.2

Acceptable damage is restricted to inelastic hinges in the columns. The foundations should, therefore, remain in their elastic range. Hence the value for the *R*-factor is taken as 1.0.

5.3.4.3 Inelastic Hinging Forces

5.3.4.3.a General

(1) Where inelastic hinging is invoked as a basis for seismic design, the force effects resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the columns has been completed utilizing the modified design forces specified in Article 5.3.4.2 of this Section as the seismic loads, The consequential forces resulting from plastic hinging shall then be used for determining design forces for most components as identified herein. The procedures for calculating these consequential forces for single column and pier supports and bents with two or more columns shall be taken as specified in the following Articles.

(2) Inelastic hinges shall be ascertained to form before any other failure due to overstress or instability in the structure and/or in the foundation. Inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired. Inelastic flexural resistance of substructure components shall be determined in accordance with the provisions of *Sections 5 (Concrete Structures)* and *Section 6 (Steel Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

(3) Superstructure and substructure components and their connections to columns shall also be designed to resist a lateral shear force from the column determined from the factored inelastic flexural resistance of the column using the resistance factors specified herein.

(4) These consequential shear forces, calculated on the basis of inelastic hinging, may be taken as the extreme seismic forces that the bridge is capable of developing.

Commentary C5.3.4.3a

(1) By virtue of Article 5.3.4.2 of this Section, alternative conservative design forces are specified if plastic hinging is not invoked as a basis for seismic design.

(2) In most cases, the maximum force effects on the foundation will be limited by the extreme horizontal force that a column is capable of developing. In these circumstances, the use of a lower force, lower than that specified in Article 5.3.4.2 of this Section, is justified and should result in a more economic foundation design.

(3) See also Appendix 4A - Overstrength Resistance, of these Specifications.

5.3.4.3.b Single Columns and Piers

(1) Force effects shall be determined for the two principal axes of a column and in the weak direction of a pier or bent as follows:

- Step l - Determine the column overstrength moment resistance. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials, the applied axial load in the column shall be determined using Extreme Event Load Combination 1, with the maximum elastic column axial load from the seismic forces determined in accordance with Article 5.2 of this Section taken as *EQ*.
- Step 2 - Using the column overstrength moment resistance, calculate the corresponding column shear force. For flared columns, this calculation shall be performed using the overstrength resistances at both the top and bottom of the flare in conjunction with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur, the column length between plastic hinges shall be used to calculate the column shear force.

(2) Force effects corresponding to a single column hinging shall be taken as:

- Axial Forces - Those determined using Extreme Event Load Combination 1, with the unreduced maximum and minimum seismic axial load of Article 5.2 of this Section taken as *EQ*.
- Moments - Those calculated in Step 1.
- Shear Force - That calculated in Step 2.

Commentary C5.3.4.3b

(1) The use of the factors 1.3 and 1.25 corresponds to the normal use of a resistance factor for reinforced concrete and structural steel columns, respectively. In this case, it provides an increase in resistance, i.e., overstrength. Thus, the term "overstrength moment resistance" denotes a factor resistance in the parlance of these Specifications.

5.3.4.3.c Piers with Two or More Columns

(1) Force effects for bents with two or more columns shall be determined both in the plane of the bent and perpendicular to the plane of the bent. When perpendicular to the plane of the bent, the forces shall be determined as for single columns in Article 5.3.4.3b of this Section. When in the plane with the bent, the forces shall be calculated as follows:

- Step 1 - Determine the column overstrength moment resistances. Use a resistance factor, ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. For both materials the initial axial load should be determined using the Extreme Event Load Combination I with *EQ* = 0.
- Step 2 - Using the column overstrength moment resistance, calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the pier. If a partial-height wall exists between the columns, the effective column height should be taken from the top of the wall. For flared columns and foundations below ground level, the provisions of Article 5.3.4.3b of this Section shall apply. For pile bents, the length of pile above the mud line shall be used to calculate the shear force.

- Step 3 - Apply the bent shear force to the center of mass of the superstructure above the pier and determine the axial forces in the columns due to overturning when the column overstrength moment resistances are developed.
- Step 4 - Using these column axial forces as *EQ* in the Extreme Event Load Combination I, determine revised column overstrength moment resistance. With the revised overstrength moment resistances, calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within ten percent of the value previously determined, use this maximum bent shear force and return to Step 3.

(2) The forces in the individual columns in the plane of a bent corresponding to column hinging shall be taken as:

- Axial Forces - The maximum and minimum axial loads determined using Extreme Event Load Combination I, with the axial load determined from the final iteration of Step 3 taken as *EQ* and treated as plus and minus.
- Moments - The column overstrength moment resistances corresponding to the maximum compressive axial load specified above.
- Shear Force - The shear force corresponding to the column overstrength moment resistances specified above, noting the provisions in Step 2 above.

Commentary C5.3.4.3c

(1) The use of the factors 1.3 and 1.25 corresponds to the normal use of a resistance factor for reinforced concrete and structural steel columns, respectively. In this case, it provides an increase in resistance, i.e., overstrength. Thus, the term "overstrength moment resistance" denotes a factor resistance in the parlance of these Specifications.

5.3.4.3.d Column and Pile Bent Design Forces

Design forces for columns and pile bents shall be taken as a consistent set of the lesser of the forces determined as specified in Article 5.3.4.1 of this Section, applied as follows:

- Axial Forces - The maximum and minimum design forces determined using Extreme Event Load Combination I with either the elastic design values determined in Article 4.2.1 taken as *EQ*, or the values corresponding to plastic hinging of the column taken as *EQ*.
- Moments - The modified design moments determined for Extreme Event Limit State Load Combination I.
- Shear Force - The lesser of either the elastic design value determined for Extreme Event Limit State Load Combination I with the seismic loads combined as specified in Article 5.2 of this Section and using an *R*-factor of 1 for the column, or the value corresponding to plastic hinging of the column.

Commentary C5.3.4.3d

The design axial forces which control both the flexural design of the column and the shear design requirements are either the maximum or minimum of the unreduced design forces or the values corresponding to plastic hinging of the columns. In most cases, the values of axial load and shear corresponding to plastic hinging of the columns will be lower than the unreduced design forces. The design shear forces are specified so that the possibility of a shear failure in the column is minimized.

When an inelastic hinging analysis is performed, these moments and shear forces are the maximum forces that can develop and, therefore, the directional load combinations of Article 5.2 of this Section do not apply.

5.3.4.3.e Pier Design Forces

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Article 5.3.4.3d of this Section and all the design requirements for columns, as specified in *Section 5 (Concrete Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later),* shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Article 5.2 of this Section, shall be applied to determine the elastic moment which is then reduced by the appropriate *R*-factor.

Commentary C5.3.4.3e

The design forces for piers specified in Article 5.3.4.3e of this Section are based on the assumption that a pier has low ductility capacity and no redundancy. As a result, a low *R*-factor of 2 is used in determining the reduced design forces, and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction, then both the design forces and, more importantly, the design requirements of Article 5.3.4.3d of this Section of these Specifications and *Section 5 (Concrete Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)* are applicable.

5.3.4.3.f Foundation Design Forces

(1) The design forces for foundations including footings, pile caps and piles may be taken as either those forces determined for the Extreme Event Load Combination I, with the seismic loads combined as specified in Article 5.2 of this Section, or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Article 5.2 of this Section.

(2) When the columns of a bent have a common footing, the final force distribution at the base of the columns in Step 4 of Article 5.3.4.3c of this Section may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

(3) When column plastic hinging is applied as the design forces for foundation, as determined in Articles 5.2 and 5.3.4.3 of this Section, additional horizontal design inertial forces applied at the center of mass of the footing or pile cap in the direction of the earthquake forces shall be applied as given in Equation 5.3.4.3.f-1.

$$F_F = \gamma_F A_S W_F \qquad (5.3.4.3.f\text{-}1)$$

where:

F_F = Additional horizontal inertial force due to seismic loading of footing or pile cap mass when column plastic hinging forces are used for the foundation design, (kN).

γ_F = Load factor for ground acceleration and shall be taken as 0.50.

A_S = Acceleration coefficient defined in Equation 3.6.2-2 ($A_S = F_{pga}\ PGA$).

W_F = Weight of footing or pile, (kN).

Commentary C5.3.4.3f

The foundation design forces specified are consistent with the design philosophy of minimizing damage that would not be readily detectable. The recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The alternate design forces are the elastic design forces. It should be noted that these may be considerably greater than the recommended design forces, although where architectural considerations govern the design of a column, the alternate elastic design forces may be less than the forces resulting from column plastic hinging.

See also the second paragraph of C5.3.4.3d of this Section.

Although the plastic hinging forces of the column would be the maximum forces transmitted to the top of the foundation (footing or pile cap), additional inertial forces are generated by the footing or pile cap due to ground excitation. During earthquake motion, the inertia forces generated to all members from the top of the footing or pile cap is translated to column plastic hinging forces which do not include the inertial forces generated at the footing or pile cap. In this case, an inertial force acting at the center of mass of the foundation level shall be added to the plastic hinging forces generated at the bottom of the column. The additional foundation inertia force shall be taken as an equivalent static force by applying the acceleration coefficient A_S, as shown in Figure C5.3.4.3f below.

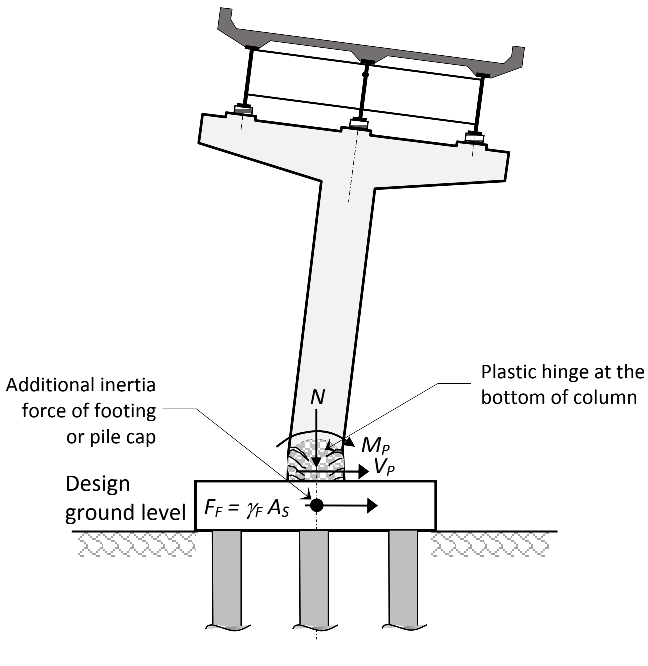


Figure C5.3.4.3f Additional Inertial Forces at Footing/Pile Cap when Applying Plastic Hinging Forces

However, when elastic forces are used in the design of foundation, no additional inertial force at the footing or pile cap will be necessary since this is already considered in the dynamic analysis or multimode analysis model.

5.3.4.4 Other Design and Detail Requirements for Seismic Performance Zones

The Seismic Performance Zones 1 to 4 in these Specifications shall comply with the other design and detail requirements of *AASHTO LRFD Bridge Design Specifications (2012)* corresponding to Seismic Zones 1 to 4.

Commentary C5.3.4.4

In these Specifications, the Seismic Performance Zones 1 to 4 corresponds to the Seismic Zones 1 to 4 of the *AASHTO LRFD Bridge Design Specifications (2012)*. In this regard, the seismic performance zones defined in this Specifications shall comply, as a minimum, with all the analysis and design requirements stipulated in the AASHTO for the different seismic zones, unless otherwise explicitly stated in these Specifications.

5.4 FOUNDATION REQUIREMENTS

5.4.1 General

(1) The provisions of this Section shall apply to the foundation requirements under the Extreme Event Limit State for earthquake effects. The design requirements for spread footings and pile foundations (which are typical foundation types in the Philippines) are given in this Section. Applicable provisions of the *AASHTO LRFD Bridge Design Specifications (2012 or later)* which are not covered in this Section shall be used for other types of foundation and limit states and for the requirements for analysis and design of foundation. Further, the AASHTO requirements for geotechnical exploration to determine the soil and rock properties (*AASHTO LRFD Bridge Design Specifications Article 2.4 and 10.4.1*) shall be applied with this Section.

(2) The design of foundation at the Extreme Event Limit State for earthquake loading shall be consistent with the requirements of Article 3.3 "Seismic Performance Requirements" of these Specifications and with the expectation that the structure collapse is prevented and that life safety is protected.

Yielding of foundation shall not be allowed under Level 1 and Level 2 Earthquake Ground Motion. Foundation yielding shall be defined as a state when the horizontal displacement at the point of superstructure inertia force tends to increase rapidly, as a result of:

- yielding of foundation members,
- yielding of the ground, or
- uplift of the foundation from the ground.

(3) The effects of seismically unstable ground prone to liquefaction, as stipulated in Section 6 of these Specifications, shall be evaluated in the analysis and design of foundation for Seismic Performance Zones 3 and 4. The potential for liquefaction should also be considered in Seismic Performance Zone 2 where loose to very loose saturated sands are within the subsurface soil profile such that

the liquefaction of these soils could impact the stability of the structure,. Reference shall also be made to the requirements of *AASHTO LRFD Bridge Design Specifications Article 10.5.4.2* for ground liquefaction assessment.

(4) The design forces for foundation shall be that determined from the requirements of Article 5.3 of this Section.

(5) When applying the provisions for pile foundation in Article 5.4.3 of these Specifications, the axial compression (bearing) resistance factors for the extreme event limit state to design foundations to resist earthquake shall be taken as 0.65. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.50 or less.

For spread footing, the resistance factors for the extreme event limit state to design foundations to resist earthquake shall be taken as 1.0.

(6) The provisions for seismic design and detail of the *AASHTO LRFD Bridge Design Specifications (2012 or* later*), Section 5 –* Concrete *Structures* shall apply to the design of foundation in these Specifications.

Commentary C5.4.1

(1) The provisions in this Section is limited to the analysis and design provisions for spread footing (with reference to the *AASHTO LRFD Bridge Design Specifications*) and for pile foundations, as stipulated in the provisions of the *Japan Road Association (JRA) Specifications for Highway Bridges, Part IV-Substructures and Part V – Seismic Design*. Other provisions for analysis and design and for other foundation types not contained in this Section shall refer to the requirements of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

(2) The design philosophy of foundation shall be consistent with the seismic performance level requirements of the bridge operational classification to minimize damage that would not be readily detectable. In this case, plastic hinges are allowed to be formed at the pier columns above the footing or pile cap level and in areas where it can readily be inspected and evaluated. However, in principle, due to the difficulty in inspection and evaluation of the foundation structure after an earthquake event, the foundation shall resist the seismic demand forces elastically without yielding or the development of plastic hinges.

When the horizontal/lateral load acting on the foundation increases, the relationship between the horizontal load and the horizontal displacement becomes nonlinear as a result of foundation member yielding or yielding of the ground. If the displacement of the foundation exceeds a certain point, it begins to increase rapidly with an increased horizontal acting force, resulting in increased damages in the foundation members with corresponding increase in residual displacement. This phenomenon can be defined as the yielding of the foundation. Therefore, yielding of the foundations, such as a caisson foundation, pile foundations, steel-pipe sheet pile foundations or diaphragm wall foundations, etc., shall be defined as a state when a rapid increase in the horizontal displacement at the point of superstructure inertia force begins, as obtained from the analysis of horizontal force and horizontal displacement relationship of the foundation.

Since it is difficult to define exactly foundation yielding because the properties of the ground often change significantly with the ground type, the yielding of foundation can be defined as a state when either of the states described in the following foundation types is reached:

1) <u>Caisson foundations</u>

Yielding of a caisson foundation can be defined as a state when one of the following occurs:

1. The caisson foundation itself yields,

2. Plastic ranges of horizontal ground reactions in the front of the foundation reaches to 60% of embedment depth of the foundation, or

3. Area of upward separation of the foundation from the ground at the bottom of the foundation reaches to 60% of total bottom area of the foundation.

2) Pile foundations

A principal factor considered to affect the yielding of foundation is the decrease in flexural stiffness when the upper limit of pile head reaction is reached. Yielding of a pile foundation can be defined as a state when one of the following occurs:

1. All the piles yield, or

2. Reaction force on the top of any piles in a row reaches the nominal bearing capacity limit.

3) *Steel-pipe sheet pile foundations*

Yielding of a steel-pipe sheet pile foundation can be defined as a state when one of the following occurs:

1. Edge stress of the steel-pipe sheet pile within 1/4 of the outer circumference of the foundation reaches its yield point,

2. Vertical ground reaction force at the bottom of more than 1/4 of the steel-pipe sheet pile reaches the ultimate bearing (compression) capacity limit,

3. Vertical reaction force at the bottom of the steel-pipe sheet pile reaching the ultimate bearing (compression) capacity plus the force causing upward separation of the foundation from the supporting ground account for about 60% of the entire steel-pipe sheet piles.

(3) The assessment for ground liquefaction potential in these Specifications is based on the *Japan Road Association (JRA) Specifications*. However, *Article 10.5.4.2* of the *AASHTO LRFD Bridge Design Specifications* gives an alternative approach to the assessment of ground liquefaction potential.

(4) It is assumed that the design of foundation complies with all the requirements of other limit states that are necessary to provide support for vertical, lateral and other loads acting on the bridge other than those due to earthquake effects.

(5) Although AASHTO specifies the resistance factor for earthquake loading to be 1.0, these Specifications recommends 0.65 as the resistance factor for foundation when Article 5.4.3 of these Specifications is applied in view of the experience of JRA on foundation design. Specifically, since the procedures for deriving the nominal pile resistance in these Specifications follows that of JRA which is based on actual site data and measurements using soil N-values, a reduced value of the resistance factor is recommended.

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor. *Article 10.5.5.2.3* of the *AASHTO LRFD Bridge Design Specifications* gives some reference on this behavior for strength limit state.

5.4.2 Spread Foundation

(1) The following provisions shall apply to foundation design for extreme event limit state under Level 2 Earthquake Ground Motion. For other limit states, the design of foundation shall comply with the requirements of *Section 10 (Foundations)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

(2) The design of foundation at the extreme event limit state for earthquake loading shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

(3) The extreme limit state design check for spread footings shall include, but not necessarily limited to:

- Bearing resistance,
- Eccentric load limitations (overturning)
- Sliding, and
- Overall stability

Verification of the above shall comply with the provisions of the *AASHTO LRFD Bridge Design Specifications* for limit state and resistance factor and for spread foundation *Section 10.5* and *10.6*, respectively.

(4) The resistance factor shall be as specified in Article 5.4.1(5) of this Section.

(5) For seismic eccentricity evaluation of foundations on soil or on rock, the location of the resultant of the reaction forces for extreme event limit state under earthquake shall be within the middle two-thirds of the base for γ_{EQ} = 0.0 and within the middle eight-tenths of the base for γ_{EQ} = 1.0. For values of γ_{EQ} between 0.0 and 1.0, the resultant location restriction shall be obtained by linear interpolation of the values given in this Article.

When γ_{EQ} = 0.5 as specified in Article 1.5 of these Specifications is applied, the eccentricity of loading shall not exceed 11/30 of the footing base.

If live loads act to reduce the eccentricity for the extreme event limit state under earthquake, γ_{EQ} shall be taken as 0.0.

Commentary C5.4.2

(1) The provisions of this Article is applicable to the verification of foundation performance under Level 2 Earthquake Ground Motion (EGM). Verification for seismic performance under Level 1 EGM is not necessary.

5.4.3 Pile Foundation

5.4.3.1 General

(1) The following provisions shall apply to foundation design for extreme event limit state under Level 2 Earthquake Ground Motion. For other limit states, the design of foundation shall comply with the requirements of *Section 10 (Foundations)* of the *AASHTO LRFD Bridge Design Specifications*

(2012 or later).

(2) The provisions given in this Article is based on the Japan Road Association requirements for pile foundation. However, the designer may also opt to use the AASHTO provisions for design of pile foundation following AASHTO requirements with the approval of DPWH.

(3) Vertical loads shall be supported solely by piles while horizontal loads shall be principally supported by piles. However, the piles and the embedded portion of the footing can jointly support the horizontal loads by carefully examining the load sharing between the members.

(4) The pile foundation shall be designed to have adequate factored axial and lateral resistance. For seismic design the effects of downdrag and lateral spreading to the soil bearing and lateral resistance shall be determined as specified in Section 6 of these Specifications, for all soil within and above the liquefiable zone, if the soil is liquefiable.

(5) The resistance factor shall be as specified in Article 5.4.1(5) of this Section.

Commentary C5.4.3.1

(1) The provisions of this Article is applicable to the verification of foundation performance under Level 2 Earthquake Ground Motion (EGM). Verification for seismic performance under Level 1 EGM is not necessary.

(2) The procedure for verification and design of pile foundations under earthquake extreme event limit state is described in this Article. Some items to be considered in pile foundation includes:

- Ground surface may be taken at the upper face of the footing when the backfill around the footing is sufficiently compacted during construction. Passive horizontal resistance around the embedded side of the footing can be expected during such time.
- Pile capacity against forces acting on the pile shall be verified considering negative skin friction and eccentric earth pressures. The group effects on the pile behavior shall be examined.
- The horizontal stability of pile foundations shall be verified in terms of displacement.

The standard design calculation for pile foundation is illustrated in Figure C5.4.3.1-1.

(3) Pile settlements and ground settlements often differ and voids can be sometimes found underneath footings of existing structures. These cases are generally found at sites where settlement occurs due to the consolidation of silt or clay intermediate layers as shown in Figure C5.4.3.1-2. Therefore, the vertical loads shall be supported exclusively by the piles.

Piles must also fully support horizontal loads. However, when backfill around the footing is sufficiently compacted up to the upper face of the footing, the design ground surface can be assumed at the upper face of the footing and the horizontal ground reaction around the embedded portion of the footing can be taken into account in the design.

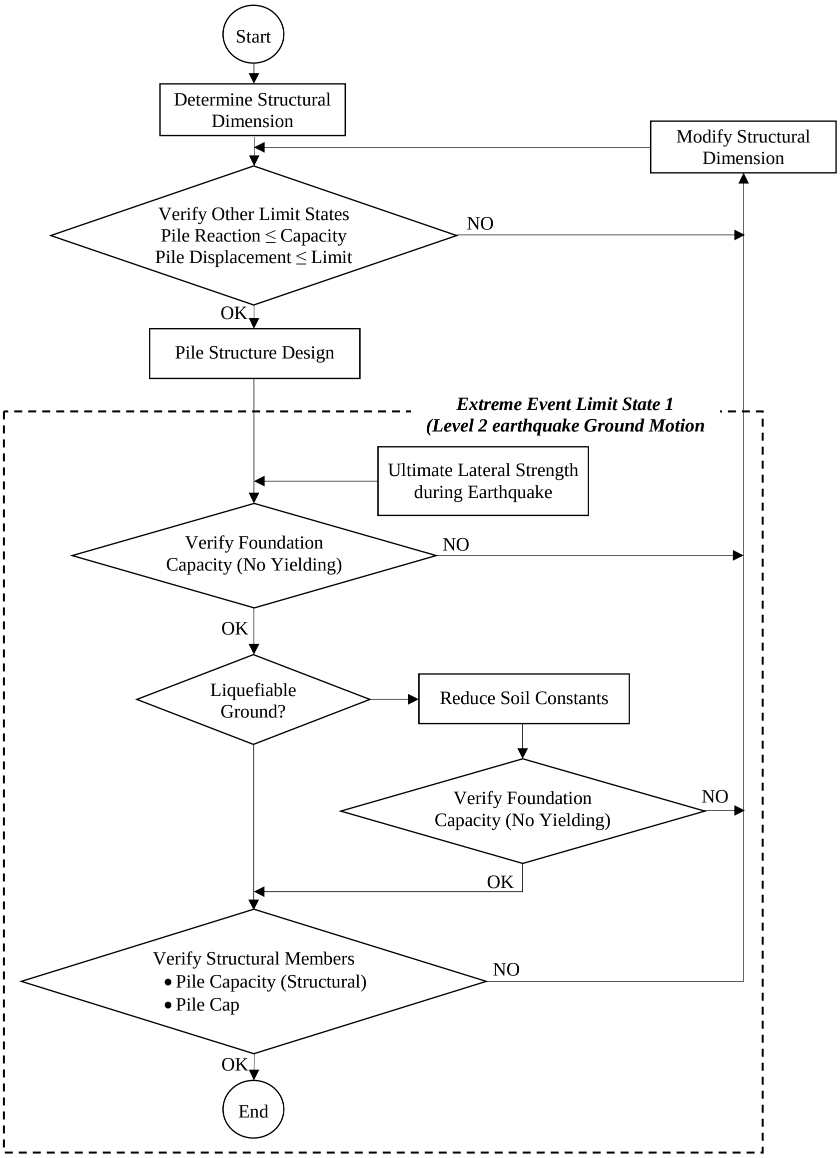


Figure C5.4.3.1-1 Pile Foundation Design Flow

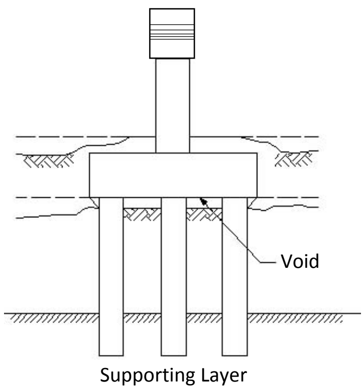


Figure C5.4.3.1-2 Generation of Voids underneath Footings

5.4.3.2 Pile Arrangement

The arrangement of piles shall be determined such that all piles will sustain uniform long-term continuous loads, in consideration of the configuration and dimensions of the abutments or piers, the dimensions and number of piles, group effects of piles, construction conditions, etc.

Commentary C5.4.3.2

This Article specifies that piles shall be arranged so that all piles will sustain uniform long-term continuous loads, in consideration of configurations and dimensions of substructures, dimensions and number of piles, group effects of piles, construction conditions, etc. Although settlement is usually evaluated from loading tests, it is difficult to estimate long-term settlements from short-term test results. Accordingly, piles shall be arranged such that each pile will sustain uniform long-term continuous loading even if installed in stable ground.

In cases when grouped piles are spaced at large intervals or pile caps are thin in comparison with the pile diameter, the footing may not be assumed as rigid structures. In such cases, the arrangement of the piles should be determined in consideration of the load distribution.

When the minimum distance between adjacent pile centers is small, the group effect of piles becomes considerably dominant, and the axial bearing capacities and coefficients of lateral subgrade reaction need to be reduced. However, in cases when the minimum distance between adjacent pile centers is larger than 2.5 times the pile diameter, group pile action can be regarded to be comparatively small, and no major problems with construction workability will occur. It should be noted that the effects of pile spacing on group pile action is related to various factors including ground type, and is not yet fully understood.

The distance between adjacent pile centers may be shorter than 2.5 times the pile diameter when the footing width has to be short due to site restrictions. In such cases, group pile action has to be fully examined in accordance with the provisions in Article 5.4.3.5 of this Section.

The distance between the outermost pile center and the footing edge shall be determined by verifying the horizontal punching shear stress and the horizontal bearing stress, and by considering the construction workability. In general, the distance between the outermost pile center and the footing edge can be 1.25 times the pile diameter for precast piles used as driven piles, bored piles, and pre-boring piles, and equal to the pile diameter for cast-in-place RC piles. Furthermore, for steel pipe soil cement piles, it can be equal to the soil cement pile diameter (refer to Figure C5.4.3.2-1).

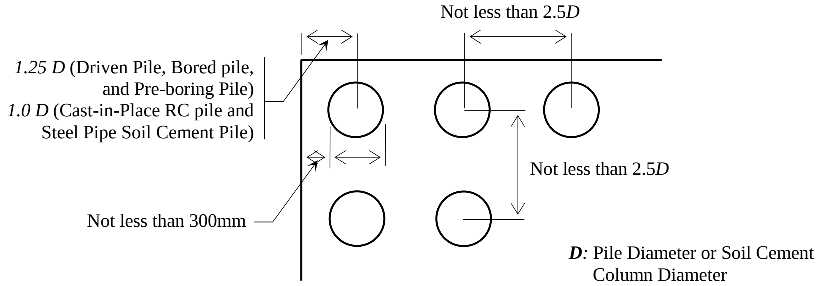


Figure C5.4.3.2-1 Minimum Distance between Pile Centers and Distance between Outermost Pile Center and Footing Edge

5.4.3.3 Nominal Axial Compression Resistance of a Single Pile (Bearing Capacity)

(1) The factored resistance of piles, R_R, shall be taken as:

$$R_R = \gamma(\varphi R_n - W_S) + W_S - W \quad \text{..........} \quad (5.4.3.3\text{-}1)$$

where:

R_R = Factored resistance of pile, (kN).

R_n = Nominal resistance of pile, (kN).

γ = Modification coefficient depending on nominal bearing resistance estimation method, shown in Table 5.4.3.3-1.

φ = Resistance factor for pile under extreme event limit state specified in Article 5.4.1(5) of this Section.

W_S = Effective weight of soil replaced by pile, (kN).

W = Effective weight of pile and soil inside pile, (kN).

However, in case of light weight pile, Equation 5.4.3.3-2 may be applied,

$$R_R = \gamma\varphi R_n \quad \text{..........} \quad (5.4.3.3\text{-}2)$$

Table 5.4.3.3-1 Modification Coefficient Depending on Nominal Bearing Capacity Estimation Method, γ

Nominal Bearing Capacity Estimation Method	Modification Coefficient, γ
Bearing Capacity Estimation Equation	1.0
Vertical Loading Test	1.2

(2) The in-situ nominal bearing capacity shall be obtained either by the empirical bearing capacity estimation formula together with adequate geotechnical investigations, or from the results of vertical loading tests.

Commentary C5.4.3.3

(1) 1) The nominal bearing resistance or capacity can be obtained either by the empirical bearing capacity estimation formula or from the results of loading tests. The nominal bearing resistance/capacity of ground is defined as the total bearing capacity including the weight of the pile, but the bearing capacity of a pile is defined as the maximum load acting on the pile head. Therefore, in case of heavy piles such as cast-in-place RC piles and steel pipe soil cement piles, Equation 5.4.3.3-1 incorporating the pile weight should be used. The weight of soils replaced by the pile W_S in Equation 5.4.3.3-1 can be expected as the bearing capacity, and therefore the resistance factor need not to be considered. Furthermore, the effective weight of the pile and soil inside the pile, W, is subtracted because the bearing capacity is evaluated at the pile head. In this case, the weight of the pile should take the value after subtracting pile buoyancy.

2) A sufficient pile bearing capacity shall be ensured by the modification coefficient (γ) for the resistance factor depending on the nominal bearing capacity estimation method given in Table C5.4.3.3-1, in addition to the resistance factor φ. The empirical bearing capacity estimation formula provides an average value based on pile load test results at various sites. However, the results of in-situ load tests will provide the bearing capacity at the site, and it is regarded that such results will be more reliable.

The results of pile load tests would provide the bearing capacity at the test site. Geographical and geological structures and the characteristics of the surrounding soils need to be carefully examined when determining the applicable area of the test results. In general, the factored bearing resistance capacity of a bridge foundation may be calculated by multiplying the nominal bearing capacity obtained from the load test (at the surrounding site) by 1.2 (modification coefficient for the nominal bearing capacity shown in Table C5.4.3.3-1), if the following conditions are satisfied:

1) The geographical and geological conditions of the surrounding ground are approximately equal to those at the foundation site.

2) The characteristics of soil resistances such as N values and compressive strength are similar at the two sites.

3) Piles are almost same in the length.

However, the modification coefficient for the factor of safety of a friction pile shall be 1.0 in consideration of long-term safety, although the short-term bearing capacity will have sufficient strength because of the neglect of end bearing capacity assumed in the design calculation.

(2) 1) The nominal bearing capacity can be obtained from the empirical bearing capacity estimation formula of Equation C5.4.3.3-1 on the basis of the results of adequate geotechnical investigations. However, it is also recommended to estimate the bearing capacity by referring to actual loading test records performed at similar sites.

$$R_n = q_d A_P + U \sum L_i f_i \qquad \text{(C5.4.3.3-1)}$$

where:

R_n = Nominal bearing capacity of pile, (kN).

A_P = Area of pile tip, (m^2).

q_d = Nominal end bearing resistance intensity per unit area, (kN/m^2).

U = Perimeter of pile, (m).

L_i = Thickness of soil layer considering shaft resistance, (m).

f_i = Maximum shaft resistance of soil layer considering pile shaft resistance, (kN/m^2).

i) Estimation of Nominal End Bearing Resistance Intensity (q_d)

(a) Driven Piles

The nominal end bearing resistance intensity q_d of a driven pile (piles installed using the driving method or the vibro-hammer method) may be evaluated from Figure C5.4.3.3-1. This figure gives the ratio of q_d to the characteristic N value at the ground at the pile tip (obtained by the method shown in Figure C5.4.3.3-2(b)) as a function of the pile embedment ratio to the supporting layer (ratio of the equivalent embedment depth above the supporting layer to the pile diameter). The equivalent embedment depth above the supporting layer can be obtained by the procedures illustrated in Figure C5.4.3.3-2. In case of open-end steel pipe piles in Figure C5.4.3.3-1, the nominal bearing resistance is reduced for an embedment ratio less than 5, in consideration of the effects of the closed tip. Furthermore, the design *N* value of the ground at the level of the pile end should be no greater than 40 in the calculation of the bearing resistance.

Figure C5.4.3.3-1 can apply to gravelly, sandy, and cohesive grounds, but not to rocks or soft rocks. The bearing capacity of driven steel pipe piles whose ends are socketed into soft rocks or mudstone may be obtained by other accepted established procedures.

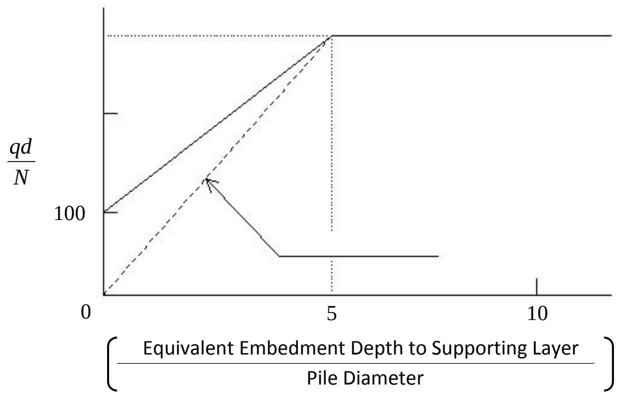
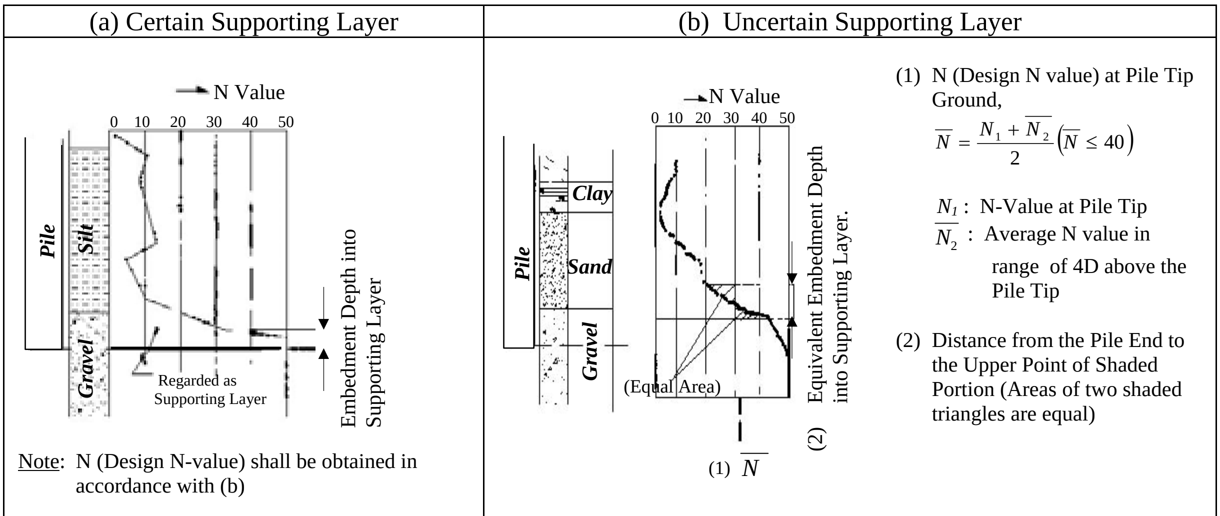


Figure C5.4.3.3-1 Evaluation Chart for Ultimate End Bearing Capacity Intensity *(q_d)*



Remarks	In determining the equivalent embedment depth into a supporting layer, soil properties in boring logs and the distribution of *N* values shall be carefully examined, and (a) or (b) in the above shall be selected.

Figure C5.4.3.3-2 Determination Method of Equivalent Embedment Depth into Supporting Layer

In Figure C5.4.3.3-1, the nominal bearing resistance is reduced for an embedment ratio less than 5. This is not to recommend that the embedment depth into a soil layer with an *N* value greater than 40 should be 5 or more times the pile diameter in case of ground similar to "(a) Certain Supporting Layer" of Figure C5.4.3.3-2. This would mean that for the case of "(b) Uncertain Supporting Layer" in Figure C5.4.3.3-2, the effects of embedment (for an embedment ratio of approximately 5) on the bearing resistance can be expected. Therefore, the design shall be carried out in consideration of field construction methods. In addition, it should be ensured that during pile installation, the piles do not sustain failure by driving them into deeper ground layer even though they have already reached a layer of sufficient bearing capacity.

The nominal end bearing resistance of piles installed by the vibro-hammer method is introduced in this Article. Since it is confirmed in the field that the results of load tests for vibro-hammer piles are approximately equal to those of conventional driven piles, both methods are jointly described in this Article as driven piles. However, the water jet technique must not be used as part of the vibro-hammer method.

(b) Cast-In-Place RC Piles

In cases of cast-in-place RC piles, the effects of ground disturbance and loosening due to pile installation works on pile bearing resistance are regarded as being greater than those in cases of driven piles. The nominal end bearing resistance intensity of cast-in-place RC piles could take the values shown in Table C5.4.3.3-1.

Since the effects of construction on the value of q_d are considerable in cases of sandy ground profile, a constant value of q_d regardless of ground strength is established for grounds with an N value of 30 or larger in the previous version of JRA Part IV. On the basis of the recent results of loading tests on cast-in-place RC piles, the nominal end bearing resistance intensity q_d may take the value of 5,000 kN/m^2, when a fully hardened sturdy gravelly ground with an N value of 50 or larger and with a thickness of 5m or greater is selected as the supporting layer.

The end bearing capacity intensity of a hard cohesive soil layer is established in accordance with that for caisson foundations.

The following points should be noted when applying Table C5.4.3.3-1:

A) The pile end shall be embedded into the hard supporting ground with an embedment length roughly equal to the pile diameter.

B) During construction, the occurrence of boiling shall be carefully noted and treated with cement slime.

C) These values shall apply to cast-in-place RC piles installed by machine excavation. The values for large diameter (ϕ =1.4~3.5m) cast-in-place RC piles installed by open excavation should be separately evaluated. Furthermore, the nominal end bearing resistance intensity can be established on the basis of the results of in-situ loading tests.

Table C5.4.3.3-1 Nominal End Bearing Resistance Intensity of Cast-In-Place RC Piles

Ground Type	Nominal Bearing Resistance End Bearing Intensity (kN/m^2)
Gravelly Layer and Sandy Layer ($N \geq 30$)	3,000
Hard Gravelly Layer ($N \geq 50$)	5,000
Hard Cohesive Soil Layer	$3q_u$

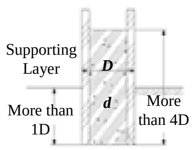
Notes: q_u : unconfined compressive strength (kN/m^2)
N : N-values from the Standard Penetration Test (SPT)

(c) Bored Pile Installation Method (Nakabori Piling Method)

The nominal end bearing resistance capacity of bored piles shall be evaluated by carefully examining the installation method.

Tip treatment procedures for precast piles (steel or concrete) by the bored pile installation method can be classified into three categories (a) to (c), and the nominal end bearing resistance intensity can be evaluated in accordance with the methods shown in Table C5.4.3.3-2.

Table C5.4.3.3-2 Nominal Bearing Resistance Intensity by the Bored Pile Installation Method, q_d

Pile Tip Treatment Method	Evaluation Methods of Nominal Bearing Resistance Intensity at Pile Tip
Final Driving Method	Evaluation Methods for Driven Piles can be applicable
Cement Milk Jetting and Mixing Method	Nominal End Bearing Resistance Intensity (kN/m^2) $q_d = \begin{cases} 150N(\leq 7{,}500), \text{ for Sandy Layer} \\ 200N(\leq 10{,}000), \text{ for Gravelly Layer} \end{cases}$ where: N: SPT N-value of Ground at Pile End
Concrete Placing Method 	Nominal End Bearing Resistance Intensity of Cast-in-place Piles can be applicable

(A) Final Driving Method

(B) Cement Milk Jetting and Mixing Method (Only Applicable to Sandy Grounds)

In the cement milk jetting and mixing method shown in Table C5.4.3.3-2, precast piles with an external diameter of between 500 to 1,000mm are normally used. The calculation methods shown in Table C5.4.3.3-2 can only apply to piles constructed by pile installation methods in which the characteristics of bearing resistance are clarified and it is confirmed that the equivalent or larger end bearing resistance in comparison with that estimated from Equation C5.4.3.3-1 can be expected from the results of previous vertical pile load tests. In addition, appropriate construction management procedures shall have been established. At that time, it is recommended that the embedment depth into the supporting layer is not shallower than the pile diameter, and the design diameter shall take the actual pile diameter. Furthermore, in cases when large-diameter precast piles whose external diameters exceed the value above, their bearing capacities and settlement characteristics shall be reviewed separately.

(C) Concrete Placing Method

This method is employed only when neither methods (a) nor (b) above apply to the treatment of the ground at the pile tip.

(d) Pre-Boring Pile Installation Method

The nominal end bearing resistance intensity by the pre-boring pile installation method shall take the values in Table C5.4.3.3-3, based on the results of past pile load tests.

Table C5.4.3.3-3 Nominal End Bearing Resistance Intensity by the Pre-Boring Pile Installation Method, q_d

Ground Type	Nominal Bearing Resistance End Bearing Intensity (kN/m^2)
Sandy Layer	150N($\leq$7,500)
Gravelly Layer	200N($\leq$10,000)

Note: N : N-value from the SPT of the ground at the pile end

Table C5.4.3.3-3 presumes that RC piles, PHC piles, or SC piles with an external diameter of between 300 and 1,000mm are used. Table C5.4.3.3-3 can only apply to piles constructed by the pile installation methods in which the characteristics of bearing capacity are clarified and it is confirmed that the equivalent or larger nominal end bearing resistance can be expected in comparison to that obtained from Equation C5.4.3.3-1 from the results of vertical loading tests. In addition, the space between the walls of the hole and the pile skin shall be backfilled with cement milk for keeping the resistance even in the small displacement level. Effective construction management procedures shall also be established. At that time, it is recommended that the embedment depth into the supporting layer is not less than the pile diameter, and the design diameter shall take the actual pile diameter. However, in cases of large-diameter piles whose external diameters exceed the value above, their bearing capacities and settlement characteristics shall be reviewed separately.

In this Article, the nominal end bearing resistance intensity by the pre-boring pile installation method is shown. Evaluation of the nominal end bearing resistance intensity of piles by this method was deemed difficult in the past, because the construction methods were variable, and data from past loading tests were limited. In this Article, however, the results of past loading tests are shown, with the limitations of the pile installation methods.

(e) Steel Pipe Soil Cement Piles

The nominal end bearing resistance intensity of piles constructed by the steel pipe soil cement pile installation method shall take the values in Table C5.4.3.3-4, based on the results of past pile load tests. Further, the area of pile tip A in Equation C5.4.3.3-1 shall be the sectional area of the soil cement column.

Table C5.4.3.3-4 Nominal Bearing Resistance Intensity at Pile Tip of Piles Installed by the Steel Pipe Soil Cement Pile Installation Method

Ground Type	Nominal Bearing Resistance Intensity (kN/m^2)
Sandy Layer	150N($\leq$7,500)
Gravelly Layer	200N($\leq$10,000)

Note: N : N-value from the SPT of the ground at the pile end

Piles installed by the steel pipe soil cement pipe installation method would generally have a soil cement column diameter of between 700 and 1,500mm, a steel pipe diameter of between 500 and 1,200mm, and soil cement cover of between 100 and 200mm. Table C5.4.3.3-4 can only apply to piles constructed by pile installation methods in which the characteristics of bearing capacity are clarified and it is confirmed that the equivalent or larger end bearing capacity in comparison with that estimated from Equation C5.4.3.3-1 can be expected from the results of previous vertical pile load tests. In addition, appropriate construction management procedures shall be established. However, in cases when steel pipe soil cement piles exceed the conditions specified above, their bearing capacities and settlement characteristics shall be reviewed separately.

Although the values in Table C5.4.3.3-4 are established in consideration of the results of loading tests on steel pipe soil cement piles, the following specifications 1) to 3) on pile tips (refer to Figure C5.4.3.3-3) need to be satisfied when applying those values.

A) The embedment depth into the supporting layer shall be roughly equal to or deeper than the diameter of the soil cement column D_{sc}.

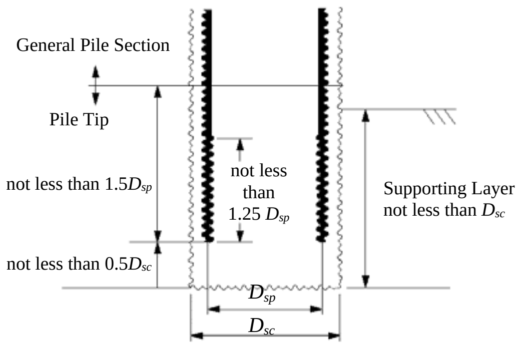


Figure C5.4.3.3-3 Pile Tip of Steel Pipe Soil Cement Pile

B) The depth of steel pipe penetrating into the soil cement column tip shall be roughly equal to or greater than 1.5 times the diameter of the steel pipe. Further, additional ribs or sections need to be installed within the range $1.25D_{sp}$ or more from the steel pipe tip.

C) The depth from the steel pipe tip to the soil cement column tip shall be roughly equal to or deeper than 0.5 times the soil cement column diameter D_{sc}.

(f) <u>Treatment of the Bored Pile Method, the Pre-boring Method, and the Steel Pipe Soil Cement Method</u>

For piles installed by the bored pile method (limited to the cement milk jetting and mixing method), the pre-boring method, and the steel pipe soil cement method, which are similar procedures and in which particular construction management procedures are needed, the empirical bearing capacity estimation formulas in this Article may apply as far as respective application conditions are satisfied. However, this particular provision shall be applicable only to piles installed by the methods in which pile head bearing capacities not less than the nominal bearing resistances obtained from Equation C5.4.3.3-1 are confirmed from the results of loading tests (three examples in cases of sandy supporting layers, and three or more examples in cases of gravelly supporting layers are necessary). Moreover, the nominal bearing resistance estimation equations shall be applicable only to grounds having characteristics similar to the site at which the loading tests were carried out.

Furthermore, for a pile installation method in which construction management procedures are not confirmed or the bearing resistance is smaller than the nominal bearing resistance obtained from Equation C5.4.3.3-1, loading tests shall be executed according to 2) below, and the adoption of the method needs to be examined by reviewing the nominal bearing resistance and construction management procedures.

ii) Estimation of Shaft Resistance Intensity f_i acting on the Pile Skin

The maximum shaft resistance intensity acting on the pile skin shall take the values shown in Table C5.4.3.3-5, depending on the pile installation method and ground type. In the case of a steel pipe soil cement pile, the perimeter of the pile *U* in Equation C5.4.3.3-1 shall take the perimeter of the soil cement column.

Note that the provisions in Commentary C.5.4.3.3-i(f) of this Section above may be applied for the maximum shaft resistance intensity of the piles installed by the bored pile method (the cement milk jetting and mixing method), the pre-boring method, and the steel pipe soil cement method.

Table C5.4.3.3-5 Maximum Shaft Resistance Intensity (kN/m^2)

Ground Type / Pile Installation Method	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-Hammer Method)	$2N$ ($\leq$ 100)	c or $10N$ ($\leq$ 150)
Cast-in-place RC pile method	$5N$ ($\leq$ 200)	c or $10N$ ($\leq$ 150)
Bored Pile Method	$2N$ ($\leq$ 100)	0.8 c or $8N$ ($\leq$ 100)
Pre-bored Pile Method	$5N$ ($\leq$ 150)	c or $10N$ ($\leq$ 100)
Steel Pipe Soil Cement Pile Method	$10N$ ($\leq$ 200)	c or $10N$ ($\leq$ 200)

Note: c : cohesion of ground (kn/m^2), N : *N*-value from SPT

The perimeter of pile *U* in Equation C5.4.3.3-1 shall be the perimeter of the cement soil pile in steel pipe soil cement Pile method.

In a soft subsoil layer with an *N*-value not larger than 2, the maximum shaft resistance shall not be evaluated using the *N*-value, because the SPT *N*-value is not a reliable indicator in such soils. In some cases, however, a certain cohesion *c* can be expected even in a ground with a small *N*-value. In such cases, it is more appropriate to estimate the maximum shaft resistance intensity by obtaining cohesion estimates from other soil tests.

For seismically unstable ground, the design shaft resistance intensity shall be obtained by multiplying the maximum shaft resistance intensity by the coefficient D_E provided in Section 6 of these Specifications.

2) The empirical bearing capacity estimation formulas shown in Commentary C.5.4.3.3-1 of this Section are established on the basis of the results of vertical pile load tests. In cases when vertical pile load tests are executed for piles of a rarely used type or installation method, the bearing resistance may be estimated by multiplying the nominal bearing capacity obtained from the pile load tests by the modification coefficient 1.2. Pile load test shall be performed on the basis of the “Method for Static Axial Compressive Load Test of Single Piles (JGS1811-2002, Japan Geological Society)” or appropriate equivalent standards as approved by DPWH.

The nominal bearing resistance of piles based on pile load tests shall be the load when the load versus settlement curve of a static loading test or equivalent becomes approximately parallel to the axis of settlement (refer to Figure C5.4.3.3-4). However, when the settlement exceeds 10% of the pile diameter when the nominal bearing resistance is mobilized, the nominal bearing resistance for design shall be replaced with the load at the settlement equal to 10% of the pile diameter (Figure C5.4.3.3-4).

When pile load tests are performed for cohesive soil ground, it is recommended to perform unconfined compression tests on undisturbed samples, static cone penetration tests, and other appropriate tests, in addition to the standard penetration tests (SPT). This will enable the establishment of a reasonable empirical bearing capacity estimation formulas. Further, the converted *N*-values should be evaluated for layers having *N*-values larger than 50.

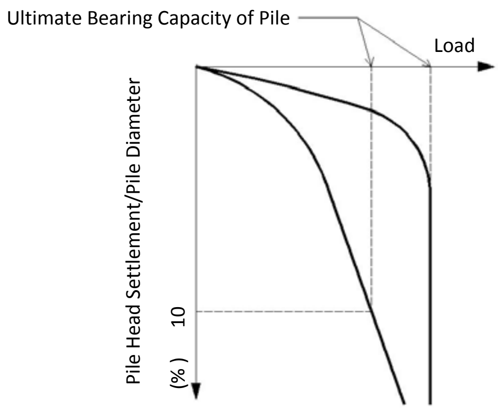


Figure C5.4.3.3-4 Evaluation of Ultimate Bearing Capacity from Vertical Pile Load Tests

5.4.3.4 Axial Pull-Out Factored Resistance of Single Pile

(1) The factored axial pull-out resistance of a single pile shall be obtained from Equation 5.4.3.4-1, considering soil conditions and construction methods.

$$P_R = \varphi P_n + W \quad \text{..........} \quad (5.4.3.4\text{-}1)$$

where:

P_R = Factored axial pull-out resistance of pile, (kN).

P_n = Nominal axial pull-out resistance of pile, (kN).

W = Effective weight of pile, (kN).

φ = Resistance factor for pile under extreme event limit state specified in Article 5.4.1(5) of this Section.

(2) The nominal axial pull-out resistance of pile shall be obtained by taking the sum of the maximum shaft resistance intensities of individual subsoil layers estimated from the results of geotechnical investigations, or by conducting tensile pile load tests.

Commentary C5.4.3.4

(1) The factored axial allowable pull-out resistance of a pile shall take the sum of the nominal axial pull-out resistance of the pile multiplied by the resistance factor and the pile weight. Here, the pile

weight should be the total weight minus the buoyancy. Further, the resistance factor need not be considered in the calculation of allowable tensile bearing capacity component generated by the pile weight.

The specification enables for an appropriate factored axial allowable pull-out resistance of a pile to be established but little data on long-term tensile pile resistance tests area available with structures sustaining considerable damage when the pile is pulled from the ground. Therefore, it is recommended to review the arrangement of the piles such that tensile forces are not generated. It is also desired to retain the stability of the foundations even when the resistance of piles likely to be pulled out is ignored.

(2) The in-situ nominal axial pull-out resistance of the pile can be evaluated from the maximum shaft resistance, which is equal to the second term of the nominal bearing capacity estimation equation (Equation C5.4.3.3-1). When the ultimate tensile bearing capacity is obtained from tensile tests, the ultimate tensile bearing capacity shall be estimated by subtracting the weight of the pile.

5.4.3.5 Consideration of Pile Group Effects

(1) The bearing capacity of a pile group subjected to axial compression forces shall be reviewed in consideration of the group effects depending on the distance between adjacent pile centers. The group effects on settlement due to axial compression forces shall also be reviewed.

(2) The bearing capacity of pile groups subjected to horizontal forces shall be reviewed in consideration of the group effects depending on the distance between adjacent pile centers.

Commentary C5.4.3.5

(1) 1) When the distance between adjacent pile centers is large, the axial bearing capacity of the pile group may be evaluated by multiplying the number of piles with the bearing capacity of a single pile. However, when the distance between adjacent pile centers is short, the bearing capacity per pile is decreased because the foundations behave as a group similar to caisson foundation consisting of the piles and soil mass located between the piles. The threshold values for the distance between adjacent pile centers changes according to the soil properties and pile allotments, and cannot be specified. Therefore, an upper limit of bearing capacity may be calculated by assuming the pile foundation as a hypothetical caisson foundation as illustrated in Figure C5.4.3.5-1, and the axial allowable bearing capacity of the pile group evaluated by Equation C5.4.3.5-1. However, in the case of piles with expanded pile-tip diameters, an additional careful investigation is needed.

$$Q_R = \varphi\,(Q_p + Q_f) \quad \text{..........} \quad \text{(C5.4.3.5-1)}$$

where:

Q_R = Factored axial bearing capacity of pile group (capacity at pile head), (kN).

φ = Resistance factor for pile under extreme event limit state specified in Article 5.4.1(5).

Q_p = Nominal end bearing capacity as pile group, (kN).

$$Q_p = A_G\, q_d{}' - W \quad \text{..........} \quad \text{(C5.4.3.5-2)}$$

A_G = Shaded area illustrated in Figure C5.4.3.5-1.

q_d' = Nominal bearing capacity intensity of soil at the base of the hypothetical caisson foundation as determined below, (kN/m^2).

W = Effective weight of soil replaced by the hypothetical caisson foundation, (kN).

Q_f = Nominal skin friction of pile group, (kN).

$$Q_f = U_G \Sigma L_i \tau_i \quad \text{(C5.4.3.5-3)}$$

U_G = Perimeter of the shaded area in Figure C5.4.3.5-1.

L_i = Thickness of each layer from the footing to the pile tip, (m).

τ_i = Shear strength intensity of each layer, (kN/m^2).

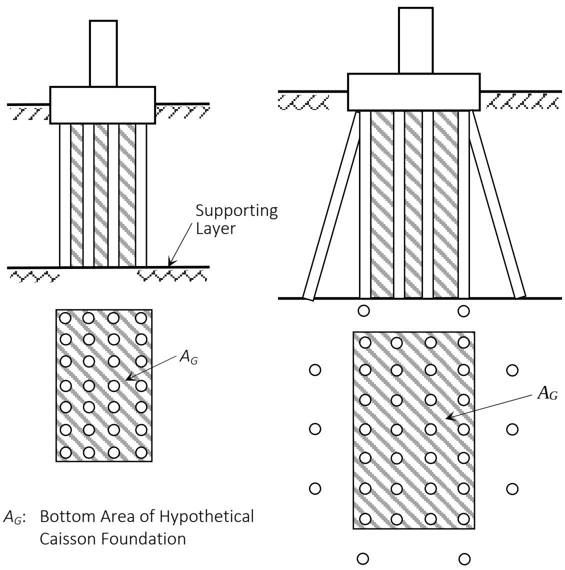


Figure C5.4.3.5-1 Hypothetical Caisson Foundation

The nominal bearing capacity intensity, q_d', may be calculated by Equation C5.4.3.5-4, as follows:

$$q_d' = \alpha c N_C + \frac{1}{2}\beta\gamma_1 B N_\gamma + \gamma_2 D_f N_q \quad \text{(C5.4.3.5-4)}$$

where:

q_d' = Nominal bearing capacity intensity of soil layer at the foundation bottom, (kN/m^2).

c = Cohesion of soil layer below the foundation bottom, (kN/m^2).

γ_1 = Unit weight of soil layer below the foundation bottom, (kN/m^3). However, use the unit weight in water at a depth lower than ground water level.

γ_2 = Unit weight of soil layer above the foundation bottom, (kN/m^3). However, use the unit weight in water at a depth lower than ground water level.

α, β = Shape factors of the foundation bottom given in Table C5.4.3.5-1.

B = Width of the foundation, (m).

D_f = Effective embedded depth from design ground surface to the bottom of foundation, (m).

N_c, N_q, N_γ = Coefficients of bearing capacity as shown in Figure C5.4.3.5-2.

Table C5.4.3.5-1 Shape Factor at Foundation Bottom

Shape of Foundation Bottom / Shape Factor	Linear	Square or Circular	Ellipse, Rectangle
α	1.0	1.3	1 + 0.3 (B/D)
β	1.0	0.6	1 - 0.4 (B/D)

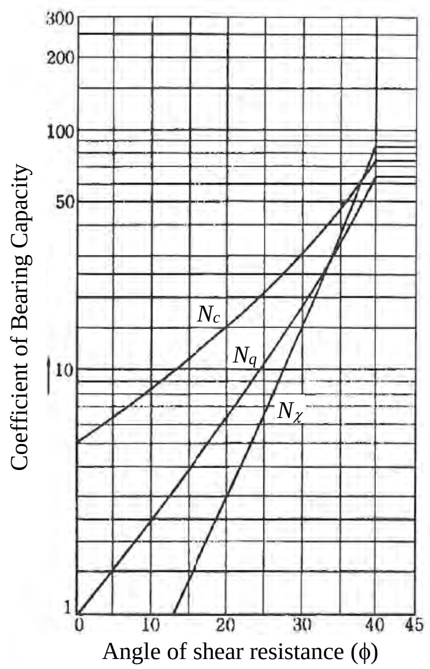


Figure C5.4.3.5-2 Coefficients of Bearing Capacity

(2) The load allotment in each pile of a pile group subjected to horizontal loads will differ due to the interference between piles. Further, the efficiency of the group is lower than that of single piles. Load allotments and group effects will vary depending on factors including ground type, pile installation method, and the number of piles. It is well-known that group-pile effects will be generated in case of short pile-to-pile distance generally not longer than *5D* (where *D* is the pile diameter). Load allotments are reduced in tracing piles and center piles in comparison to leading piles and outer piles. Furthermore, the efficiency of the group decreases significantly with decrease in pile-to-pile distance.

However, it is difficult to determine group effects considering the various factors mentioned above. Moreover, the difference in load allotments to individual piles can be covered within the range of the design safety factor. However, the coefficients of horizontal subgrade reaction for grouped piles are practically assumed to be identical to those for single piles, in case of a pile-to-pile distance of approximately *2.5D.* The reason for this is that differences in the horizontal coefficients of the subgrade reaction will not have a significant effect on pile stress.

However, in case when significantly shorter distances are likely to occur, a decrease in design vertical bearing capacity needs to be considered. To evaluate the reduced coefficient, the coefficient of horizontal subgrade reaction obtained from Article 4.4.2 (Equation C.4.4.2-4) of these Specifications may be multiplied by the modification coefficient (μ) of Equation C5.4.3.5-5.

$$\mu = 1 - 0.2\left(2.5 - \frac{L}{D}\right) \quad [L<2.5D] \quad \text{(C5.4.3.5-5)}$$

where:

μ = Modification coefficient for the coefficient of horizontal subgrade reaction.

L = Distance between adjacent pile centers, (m).

D = Pile diameter equal to the diameter of the soil cement column in cases of steel soil cement pile, (m).

However, it has been shown from recent experiments on both models and actual pile groups that group effects cannot be neglected for piles subjected to large deformations. Accordingly, in the verification for the Level 2 Earthquake Ground Motion condition, the coefficient of horizontal subgrade reaction must be reduced even for piles with a pile-to-pile distance not less than 2.5 times the pile diameter (see Section 6 of these Specifications for reduction of geotechnical parameters).

5.4.3.6 Pile Axial Spring Constants for Design

The axial spring constant K_V of a single pile for use in design shall be estimated from the empirical formula derived from the vertical pile loading test and the results of soil test, or from load-settlement curves from vertical loading test (refer to Commentary C5.4.3.6 of this Section). The pile axial spring shall be applied at the pile head.

Commentary C5.4.3.6

(1) The axial spring constant of a pile K_V is defined as the axial force capable of generating a unit displacement at the pile head in the longitudinal direction of the pile. This spring constant is used to estimate elastic settlement of the pile foundation, as well as to estimate the pile reaction specified in this Section. When K_V is used in the evaluation of the pile head reaction, its value will affect the pile head moment and axial tensile force. In particular, since footings having a short pile-to-pile distance and few pile rows tend to rotate, the pile head moment and other forces will significantly affect the value of K_V. In such cases, the properties of K_V should be fully investigated and an appropriate value adopted.

Although it is recommended to obtain the value of K_V from the pile head load and pile head settlement curves from vertical pile load tests, it may be obtained from empirical estimation formulas similar to the case of bearing capacity.

1) When the value of K_V is estimated based on pile load tests, the value of a in Equation C5.4.3.6-1 was derived from the values of K_V measured from numerous pile load test results. The value of a is evaluated in terms of the embedment ratio *(L/D)* depending on the pile installation procedure, as indicated in Equation C5.4.3.6-3.

$$K_V = a\frac{A_P E_p}{L} \quad \text{(C5.4.3.6-1)}$$

where:

K_V = Axial spring constant of pile, (kN/m).

a = Proportional coefficient.

A_p = Net cross-sectional area of pile, (mm^2).

E_p = Young's modulus of pile, (kN/mm^2).

L = Pile length, (m).

D = Pile diameter, (m).

Furthermore, the axial spring constant K_V of a steel pipe soil cement pile is evaluated from Equation C5.4.3.6-2.

$$K_V = a\frac{A_{sp}E_{sp} + A_{sc}E_{sc}}{L} \qquad \text{(C5.4.3.6-2)}$$

where:

A_{sp} = Net cross-sectional area of steel pipe, (mm^2).

E_{sp} = Young's modulus of steel pipe, (kN/mm^2).

A_{sc} = Net cross-sectional area of soil cement column, (mm^2).

E_{sc} = Modulus of deformation of soil cement, (kN/mm^2).

$E_{sc} = 500\, q_u$

q_u = Unconfined compressive strength of soil cement, (kN/mm^2).

L = Pile length, (m).

a values can be evaluated from Equation C5.4.3.6-3 below.

$$\left.\begin{array}{lll}
a = 0.014\,(L/D) + 0.72 & : \text{Driven Piles (Blow Method)} \\
a = 0.017\,(L/D) - 0.014 & : \text{Driven Piles (Vibro-Hammer Method)} \\
a = 0.031\,(L/D) - 0.15 & : \text{Cast-in-place RC piles} \\
a = 0.010\,(L/D) + 0.36 & : \text{Bored Piles} \\
a = 0.013\,(L/D) + 0.53 & : \text{Pre-Boring Piles} \\
a = 0.040\,(L/D) + 0.15 & : \text{Steel Pile Soil Cement Piles}
\end{array}\right\} \qquad \text{(C5.4.3.6-3)}$$

The results of loading tests were analyzed under the following conditions:

a) The measured K_V value is taken as the secant gradient at the yield point judged from log*P*-log*S* method on the *P-S* curve from loading tests, where *P* is pile head load and *S* is pile head settlement.

b) As most data in Equation C5.4.3.6-3 are for an embedment ratio $L/D \geq 10$, Equation C5.4.3.6-3 should generally apply to piles with an embedment ratio not less than 10. Therefore, for piles with $L/D < 10$, it is recommended to determine K_V by referring to the results of pile load tests performed under similar conditions.

c) The value of A_p is the net cross-sectional area.

In case of a steel pipe soil cement pile, D denotes the diameter of the soil cement column. Moreover, when an SC pile is used as the upper pile of a PHC pile, a, A_p and E_p shall take the values of the PHC pile.

(2) When the value of K_V is estimated from soil tests, it is evaluated by assuming an elastic pile with distributed springs on its perimeter and a spring at the pile end. K_v can be estimated by substituting a from Equation C5.4.3.6-4 into Equation C5.4.3.6-1 or Equation C5.4.3.6-2. K_V can be estimated from Equation C5.4.3.6-4, if coefficients C_S and K_V *are* given.

$$a = \frac{\lambda \tanh \lambda + \gamma}{\gamma \tanh \lambda + \lambda} \cdot \lambda \qquad \text{(C5.4.3.6-4)}$$

where:

$$\gamma = \frac{A_i \cdot k_V \cdot L}{A_p E_p}$$

$$\lambda = L\sqrt{\frac{C_S U}{A_p E_p}}$$

A_i = Block area at pile tip, (m^2).

U = Pile perimeter, (m).

K_v = Vertical coefficient of subgrade reaction at pile end, (kN/m^3).

C_S = Coefficient of sliding between pile shaft and surrounding soil, (kN/m^3).

(3) When a different pile installation/construction method, other than that mentioned in this Article, is employed, an empirical formula to estimate K_V value may be proposed following a careful review of the ground condition and construction management procedures.

5.4.3.7 Calculation of Pile Reaction and Displacements

(1) Pile reactions and displacements shall be evaluated considering the properties of the pile structure and the ground. The foundation structure shall be represented by a rigid pile cap with the pile and the ground represented by a linear elastic structure with spring constants in the axial, lateral and rotational directions. The spring constants are modeled as either lumped springs or discrete elements with nodal springs as illustrated in Figure C4.4.3-2 of Article 4.4.3 of these Specifications.

(2) When discrete elements with nodal springs are used to calculated pile reactions and displacements as illustrated in Figure C4.4.3-2(c), the horizontal spring K_{HP} shall be determined using the k_{H0} evaluated from Equation C.4.4.2-5.

(3) When lumped springs are used to model the pile foundation as illustrated in Figure C4.4.3-2(b), the spring constants A_{ss}, A_{rr}, A_{sr}, and A_{rs}, shall be determined using the K_1–K_4 values from Table C4.4.3-2 using the k_{H0} evaluated from Equation C.4.4.2-5.

Commentary C5.4.3.7

(1) When checking the extreme event limit state under earthquake, the pile may be treated as an elastic body since yielding of the foundation is not allowed. In this regard, the coefficients of horizontal subgrade reactions as well as the pile behavior are considered to be linear. (see Figure 5.4.3.7-1).

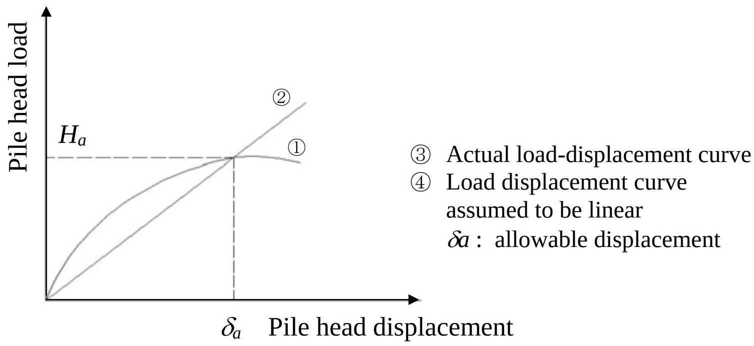


Figure 5.4.3.7-1 Assuming Linearity of Pile Behaviors

The following calculation method considers piles and soil layers to be linearly elastic - using a rigid-frame model in which the pile head is connected to the pile cap thus making the pile a beam fixed to an elastic floor. The displacement method is used to solve the equilibrium equation for the whole pile foundation model with horizontal and vertical forces and rotational moment acting on the pile foundation while assuming the pile cap as a rigid body with spring matrix at the pile head.

Calculation based on a displacement method.

1) Assumptions in the displacement method.

For computational convenience, pile reaction and footing displacement are assumed as follows:

(a) The pile foundation is assumed to be a two-dimensional structure

(b) Piles are linearly elastic in bearing/push-in, tension/pull-out and bending displacements. Both axial and radial spring constants in both the axial and lateral direction at the pile head are constant, irrespective of loads.

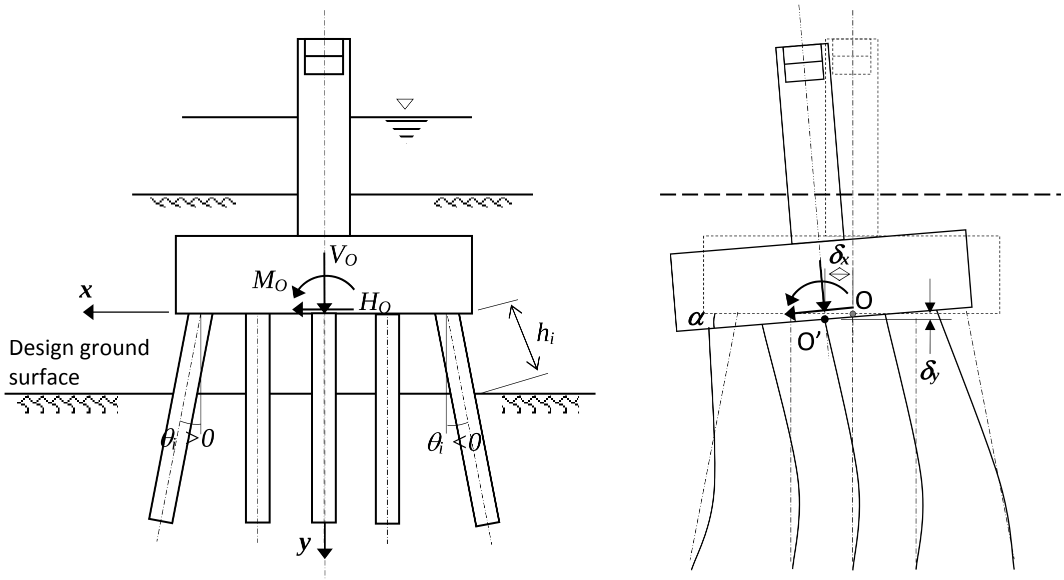
The same spring constant is applied to bearing/push-in and tension/pull-out displacements.

(c) Footings are rigid and rotates around the centroid of group piles.

2) Calculation Method

In the displacement method, the coordinate is formed as shown in Figure 5.4.3.7-2 with the origin set at an arbitrary point O of the foundation. As illustrated in the Figure, the external forces act at point O and the displacements δ_x, δ_y and rotation α set at point O in the direction of the coordinate axis.

The origin O may be selected from any arbitrary points, but it is recommended to coincide it with the centroid of the pile group below the pile cap/footing.



b. Substructure Layout a. Deformed Shape

Figure 5.4.3.7-2 Coordinate in a calculation using the Displacement Method

In this case, the displacement of the origin can be obtained by solving the following simultaneous equation with three unknowns:

$$\left.\begin{aligned} A_{xx}\cdot\delta_x + A_{xy}\cdot\delta_y + A_{x\alpha}\cdot\alpha &= H_0 \\ A_{yx}\cdot\delta_x + A_{yy}\cdot\delta_y + A_{y\alpha}\cdot\alpha &= V_0 \\ A_{\alpha x}\cdot\delta_x + A_{\alpha y}\cdot\delta_y + A_{\alpha\alpha}\cdot\alpha &= M_0 \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(C5.4.3.7-1)}$$

Assuming that the footing bottom is horizontal, each coefficient can be obtained by using the following equations:

$$\left.\begin{aligned} A_{xx} &= \sum\left(K_1\cdot\cos^2\theta_i + K_V\cdot\sin^2\theta_i\right) \\ A_{xy} = A_{yx} &= \sum\left(K_V - K_1\right)\cdot\sin\theta_i\cdot\cos\theta_i \\ A_{x\alpha} = A_{\alpha x} &= \sum\left\{\left(K_V - K_1\right)x_i\cdot\sin\theta_i\cdot\cos\theta_i - K_2\cdot\cos\theta_i\right\} \\ A_{yy} &= \sum\left(K_V\cdot\cos^2\theta_i + K_1\cdot\sin^2\theta_i\right) \\ A_{y\alpha} = A_{\alpha y} &= \sum\left\{\left(K_V\cdot\cos^2\theta_i + K_1\cdot\sin^2\theta_i\right)x_i + K_2\cdot\sin\theta_i\right\} \\ A_{\alpha\alpha} &= \sum\left\{\left(K_V\cdot\cos^2\theta_i + K_1\cdot\sin^2\theta_i\right)x_i^2 + \left(K_2 + K_3\right)x_i\cdot\sin\theta_1 + K_4\right\} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots \text{(C5.4.3.7-2)}$$

where:

H_0 = lateral loads acting at the bottom of pile cap (kN)

V_0 = vertical loads acting at the bottom of pile cap (kN)

M_0 = moment (external force) at the origin O (kN·m)

δ_x = lateral displacement from the origin O (m)

δ_y = vertical displacement from the origin O (m)

α = rotational angle of the footing at the origin O (rad)

x_i = x-coordinate of the i-th pile head (m)

θ_i = angle from the pile vertical axis of the i-th pile (degree). Sign to be in accordance with Figure 5.4.3.7-2.

If the coefficients of horizontal subgrade reaction are constant irrespective of depths, the spring constants in the lateral direction K_1, K_2, K_3 and K_4 of the pile can be obtained by using Figure C4.4.3-3 or Table C4.4.3-2.

The displacements (δ_x, δ_y, and α) below are derived by solving Equations C5.4.3.7-1 and C5.4.3.7-2:

$$\left.\begin{aligned} \delta_x &= \frac{H_0 \bullet A_{\alpha\alpha} - M_0 \bullet A_{x\alpha}}{A_{xx} \bullet A_{\alpha\alpha} - A_{x\alpha} \bullet A_{\alpha x}} \\ \delta_y &= \frac{V_0}{A_{yy}} \\ \alpha &= \frac{-H_0 \bullet A_{\alpha x} + M_0 \bullet A_{xx}}{A_{xx} \bullet A_{\alpha\alpha} - A_{x\alpha} \bullet A_{\alpha x}} \end{aligned}\right\} \quad \text{.......................................} \quad \text{(C5.4.3.7-3)}$$

By using the displacements (δ_x, δ_y, α) at the footing origin O obtained from the results of the above-mentioned calculations, the pile axial force P_{Ni}, pile radial force P_{Hi}, and moment M_{ti} acting on each pile head can be obtained by using the following equations:

$$\left.\begin{aligned} P_{Ni} &= K_V \bullet \delta_{yi}' \\ P_{Hi} &= K_1 \bullet \delta_{xi}' - K_2 \bullet \alpha \\ M_{ti} &= -K_3 \bullet \delta_{xi}' + K_4 \bullet \alpha \end{aligned}\right\} \quad \text{.......................................} \quad \text{(C5.4.3.7-4)}$$

$$\left.\begin{aligned} \delta_{xi}' &= \delta_x \bullet \cos\theta_i - (\delta_y + \alpha x_i) \bullet \sin\theta_i \\ \delta_{yi}' &= \delta_x \bullet \sin\theta_i + (\delta_y + \alpha x_i) \bullet \cos\theta_i \end{aligned}\right\} \quad \text{..............................} \quad \text{(C5.4.3.7-5)}$$

where:

δ_{xi}' = Radial displacement at the i-th pile head, (m).

δ_{yi}' = Axial displacement at the i-th pile head, (m).

K_V = Pile axial force which generates a unit volume of axial displacement at the pile head (pile’s axial spring constant, (kN/m).

K_1, K_2, K_3 and K_4 = Spring constants of the pile in the lateral direction.

x_i = x-coordinate of the i-th pile head, (m).

θ_i = Vertical axis angle from the i-th pile axis for battered pile, (degree).

P_{Ni} = Axial force of the i-th pile, (kN).

P_{Hi} = Radial force of the i-th pile, (kN).

M_{ti} = Moment as external force acting on the i-th pile head, (kN-m).

Among the values obtained from the above, M_{ti} represents the external moment force distributed at the pile head, and the bending moment M_{bi} as the internal moment force at the pile head with an opposite sign (i.e., $M_{bi} = M_{ti}$).

Moreover, the pile head vertical reaction V_i and horizontal reaction H_i are given by the following equations which are used to calculate the reinforcement arrangement of footings:

$$\left.\begin{aligned} V_i &= P_{Ni} \cdot \cos\theta_i - P_{Hi} \cdot \sin\theta_i \\ H_i &= P_{Ni} \cdot \sin\theta_i + P_{Hi} \cdot \cos\theta_i \end{aligned}\right\} \quad \text{(C5.4.3.7-6)}$$

Since the following equation must hold effective, it helps the designers to check whether or not the calculating process is correct:

$$\left.\begin{aligned} &\sum H_i = H_0 \\ &\sum V_i = V_0 \\ &\sum (M_{ti} + V_i \cdot x_i) = M_0 \end{aligned}\right\} \quad \text{(C5.4.3.7-7)}$$

3) Vertical Piles with Symmetrical Arrangement

When the piles are set vertically with symmetrical arrangement (typical layout of pile foundation with n piles), Equation C5.4.3.7-3 is simplified using the spring constants K_1, K_2, K_3 and K_4 and K_v into:

$$\left.\begin{aligned} \delta_x &= \frac{H_0 + \dfrac{nK_2}{K_V \sum x_i^2 + nK_4} M_0}{nK_1 - \dfrac{(nK_2)^2}{K_V \sum x_i^2 + nK_4}} \\ \delta_y &= \frac{V_0}{nK_V} \\ \alpha &= \frac{M_0 + \frac{1}{2}\lambda H_0}{K_V \sum x_i^2 + n\left(K_4 - \dfrac{K_2^2}{K_1}\right)} \end{aligned}\right\} \quad \text{(C5.4.3.7-8)}$$

$$\left.\begin{aligned} P_{Ni} &= \frac{V_0}{n} + \frac{M_0 + \frac{1}{2}\lambda H_0}{\sum x_i^2 + \dfrac{n}{K_V}\left(K_4 - \dfrac{K_2^2}{K_1}\right)} x_i \\ P_{Hi} &= \frac{H_0}{n} \\ M_{ti} &= \frac{1}{n}\left(M_0 - \sum P_{Ni} \cdot x_i\right) \end{aligned}\right\} \quad \text{(C5.4.3.7-9)}$$

where:

$$\lambda = h + \frac{1}{\beta}$$

$\frac{1}{2}\lambda H_0 = 0$, in case of a hinged frame.

5.4.3.8 Design of Piles Against Loads After Construction

(1) The axial forces in pile sections due to axial bearing/push-in or tensile forces/pull-out shall be evaluated in consideration of the ground properties.

(2) The bending moments and shear forces in pile sections due to lateral forces and pile head moments shall be calculated by modeling the pile structure as a beam on an elastic foundation.

(3) The safety of pile sections against axial forces, bending moments, and shear forces shall be verified.

Commentary C5.4.3.8

(1) 1) Generally, the axial compression/push-in force in a pile body reduces in the direction of its depth, as illustrated in Figure C.5.4.3.8-1(b), but it is common to assume the axial compression force to be constant in the depth direction.

2) Also, it can be assumed that the axial tension/pull-out force varies linearly from the pile tip to the head as illustrated in Figure C.5.4.3.8-1(c) with the pile tip stress being zero, when the soil layers are uniform. When the upper layer is soft, the axial force may be calculated by assuming that it does not change in the soft ground as illustrated in Figure C.5.4.3.8-1(d).

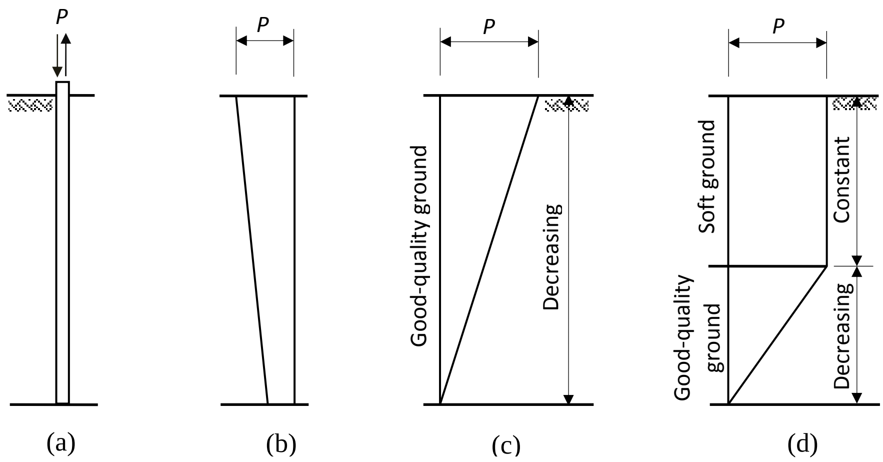


Figure C.5.4.3.8-1 Axial Force Assumptions

(2) There are two types of external forces applied at the pile head which generates lateral displacement - lateral shear force and pile head moment generated when the pile and the pile cap are designed with a rigid connection. This commentary provides a method to design the pile body to resist these external forces by assuming that the pile is a beam on an elastic floor.

Since the pile head reaction should generally be obtained by considering the footing displacement, its influences should be taken into account when designing the pile body.

1) Basic Assumption for Designing the Pile Body

It is desirable that design bending moment of the pile body should be determined by taking the following two points into consideration:

① For a rigidly connected pile head, the bending moment to be used in the design of the pile head portion shall be the larger of the pile head bending moment calculated by the displacement method or the maximum bending moment below the ground obtained when the pile head connection is taken as hinged.

② The pile body middle segment shall be designed by using the larger value between bending moments for the case of the rigid connection or hinge connection between the pile head and the pile cap, even if the rigid connection is used.

The above provisions are recommended since it is difficult to secure during construction the ideal connection condition in design, even when the pile is designed with a rigid connection with the pile cap. Moreover, it is more conservative to consider the worst scenario during an earthquake when the pile head becomes a hinge connection.

2) Computation Classification by Conditions of Piles

① When the coefficient of horizontal subgrade reaction of the pile is uniform and if its embedded depth is 3/β or more, the design calculations may be carried out by assuming that the pile is a beam of semi-infinite length with a constant coefficient of horizontal subgrade reaction.

② When the coefficient of horizontal subgrade reaction of the pile is uniform and if it's embedded depth is less than 3/β, the design calculations may be carried out by assuming that the pile is a beam of finite length with a constant coefficient of horizontal subgrade reaction.

③ When the coefficient of horizontal subgrade reaction is not uniform, the design calculations may be carried out in accordance with Item 1 or 2 above, by using an average value of coefficients of horizontal subgrade reaction from the ground surface to the depth of 1/β.

i. The calculation formulas for a pile of semi-infinite length

The calculation formulas are shown in Table C.5.4.3.8-1a and C.5.4.3.8-1b. In Table C.5.4.3.8-1(a) and (b), apply Case a) as the basic calculation formula in case of a rigid pile head connection, and Case b) for calculation in case of a hinged connection as specified in Item 1) above.

ii. Calculation formulas for a pile of finite length

When the pile length is less than 3/β the conditions of the pile tip may affect the forces, and the results to be obtained by using the calculation formulas for semi-infinite length may sometimes lead to large errors. In such case, calculation formulas for finite length is recommended. When the βL_e ranges from 2.5 to 3, the calculation formulas for semi-infinite length may be used, because such errors are not so large. Hence, it is better to use the following calculation formulas only for piles with the βL_e being less than 2.5.

Since it is rather difficult to indicate displacement, bending moments, and shear forces for a pile of finite length with a single formula, only the basic formulas are shown below (see Figure C.5.4.3.8-2).

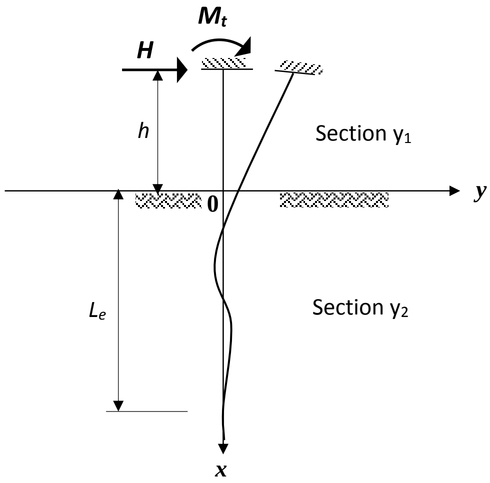


Figure C.5.4.3.8-2 Calculation Coordinates

The deflection curve for the section above the ground is represented by the following equation as:

$$y_1 = a_1x^3 + a_2x^2 + a_3x + a_4 \quad \text{(C5.4.3.8-1)}$$

Also, the deflection curve for the section within the ground is given as:

$$y_2 = e^{\beta x}(C_1\cos\beta_x + C_2\sin\beta_x) + e^{-\beta x}(C_3\cos\beta_x + C_4\sin\beta_x) \quad \text{(C5.4.3.8-2)}$$

Since the above-mentioned equations contain eight unknown values, from a_1 to a_4 and from C_1 to C_4, they may be solved by using the following boundary conditions:

$$\left.\begin{aligned} EI\,[y_1^{(3)}]_{x=-h} &= H \\ EI\,[y_1^{(2)}]_{x=-h} &= M_t \end{aligned}\right\} \text{Pile head conditions} \quad \text{(C5.4.3.8-3)}$$

$$\left.\begin{aligned} [y_1]_{x=0} &= [y_2]_{x=0} \\ [y_1^{(1)}]_{x=0} &= [y_2^{(1)}]_{x=0} \\ [y_1^{(2)}]_{x=0} &= [y_2^{(2)}]_{x=0} \\ [y_1^{(3)}]_{x=0} &= [y_2^{(3)}]_{x=0} \end{aligned}\right\} \text{Continuous conditions when } x=0 \quad \text{(C5.4.3.8-4)}$$

The conditions at the pile tip (where $x = L_e$) can be classified into the three cases. Any of the following appropriate case should be adopted.

(a) When considering the pile tip as free,

$$\left.\begin{aligned} [y_2^{(2)}]_{x=L_e} &= 0 \\ [y_2^{(3)}]_{x=L_e} &= 0 \end{aligned}\right\} \quad \text{(C5.4.3.8-5)}$$

(b) When considering the pile tip as hinged,

$$\left.\begin{aligned}[y_2]_{x=Le} &= 0\\ [y_2^{(2)}]_{x=Le} &= 0\end{aligned}\right\} \quad \text{..........} \quad (C5.4.3.8\text{-}6)$$

(c) When considering the pile tip as fixed,

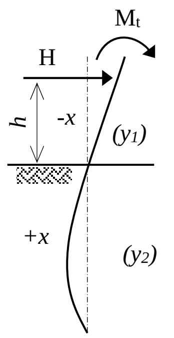
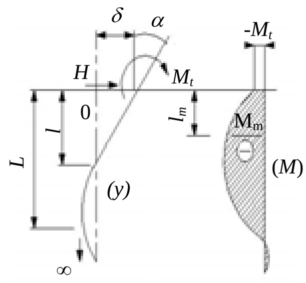
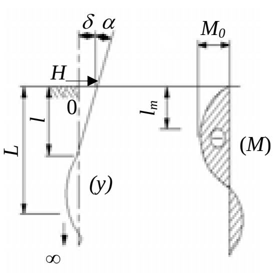
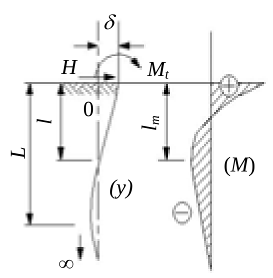
$$\left.\begin{aligned}[y_2]_{x=Le} &= 0\\ [y_2^{(1)}]_{x=Le} &= 0\end{aligned}\right\} \quad \text{..........} \quad (C5.4.3.8\text{-}7)$$

The whole calculation formulas to be obtained under these boundary conditions are shown in Table C.5.4.3.8-1a, C.5.4.3.8-1b and Table C.5.4.3.8-2.

(3) This article provides that no influence of buckling is taken into account for piles whose whole length is embedded in the ground, because buckling is constrained even if the soil layers at the lateral sides are soft.

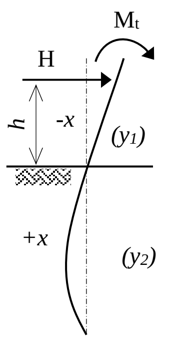
When the pile is constructed with its upper end projecting above the ground (referring to the seismic design ground surface as presented in Article 3.5.2 of these Specifications), buckling sometimes govern the determination of the pile cross section.

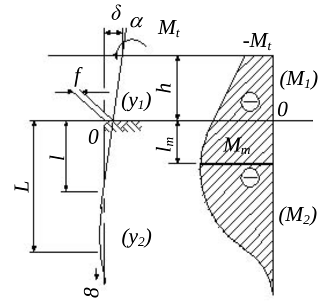
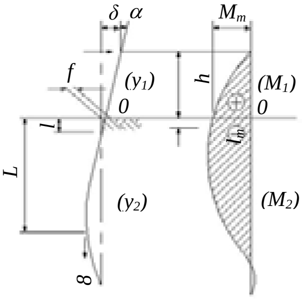
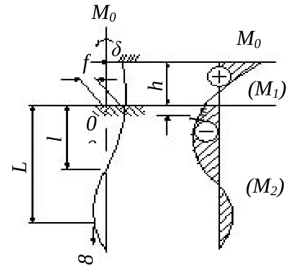
Table C.5.4.3.8-1a General Formula for Lateral Forces and Moments as External Forces (1) for Pile Embedded in the Ground

<table>
<tr><td>Differential equations of deflection curve</td><td></td><td>Portion above the ground: $EI\dfrac{d^4y_1}{dx^4}=0$
Portion in the ground: $EI\dfrac{d^4y_2}{dx^4}+p=0$
$p=k_H D y_2$
H : lateral force of a pile (N)
M_t : moment as external force at the pile head (N-mm)
D : pile diameter (mm)
E : Young's modulus of the pile (N/mm^2)</td><td>I: moment of inertia of the pile cross section (mm^4)
k_H: coefficient of horizontal subgrade reaction (N/mm^3)
h: height above the ground surface where H and M_t act (mm)
$\beta = \sqrt[4]{k_H D/4EI}$ (mm^{-1})
$h_0 = \dfrac{M_t}{H}$ (mm)</td></tr>
<tr><td>Pile condition</td><td colspan="3">Pile Embedded in the Ground (h=0)</td></tr>
<tr><td>Deflection curve and bending moment diagram</td><td>(a) Basic system
</td><td>(b) When M_t = 0 (h_0 = 0)
</td><td>(c) When pile head does not rotate
</td></tr>
</table>

a	Deflection curve, *y* (mm)	$y = \frac{H}{2EI\beta^3} e^{-\beta x}\left[(1+\beta h_0)\cos\beta_x - \beta h_0 \sin\beta_x\right]$	$y = \frac{H}{2EI\beta^3} e^{-\beta x} \cos\beta_x$	$y = \frac{H}{4EI\beta^3} e^{-\beta x}\left[\cos\beta_x + \sin\beta_x\right]$
b	Displacement of pile head, δ (mm)	$\delta = \frac{H}{2EI\beta^3} + \frac{M_t}{2EI\beta^2} = \frac{1+\beta H_0}{2EI\beta^3} H$	$\delta = \frac{H}{2EI\beta^3}$	$\delta = \frac{H}{4EI\beta^3} = \frac{\beta H}{k_H D}$
c	Displacement of ground surface, *f*(mm)	$f = \delta$	$f = \delta$	$f = \delta$
d	Pile head inclination angle, α (rad)	$\alpha = \frac{H}{2EI\beta^2} + \frac{M_t}{EI\beta} = \frac{1+2\beta h_0}{2EI\beta^3} H$	$\alpha = \frac{H}{2EI\beta^2}$	$\alpha = 0$
e	Bending moment at each section of the pile, *M* (N-mm)	$M = -\frac{H}{\beta} e^{-\beta x}\left[\beta h_0 \cos\beta_x + (1+\beta h_0)\sin\beta_x\right]$	$M = -\frac{H}{\beta} e^{-\beta x} \sin\beta_x$	$M = -\frac{H}{2\beta} e^{-\beta x}(\sin\beta_x - \cos\beta_x)$
f	Shear force at each section of the pile, *S* (N)	$S = -He^{-\beta_x}\left[\cos\beta_x - (1+2\beta h_0)\sin\beta_x\right]$	$S = -He^{-\beta_x}(\cos\beta_x - \sin\beta_x)$	$S = -He^{-\beta_x}\cos\beta_x$
g	Bending moment at pile head, M_0 (N-mm)	$M_0 = -M_t = -Hh_0$	$M_0 = 0$	$M_0 = \frac{H}{2\beta}$
h	Bending moment at a point along embedded section, M_m (N-mm)	$M_m = -\frac{H}{2\beta}\sqrt{(1+2\beta h_0)^2 + 1} \bullet \exp(-\beta l_m)$	$M_m = -\frac{H}{\beta} e^{-\frac{\pi}{4}} \sin\frac{\pi}{4} = -0.3224\frac{H}{\beta}$	$M_m = -\frac{H}{2\beta} e^{-\frac{\pi}{2}} = -0.2079 M_0$
i	l_m (mm)	$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0}$	$l_m = \frac{\pi}{4\beta}$	$l_m = \frac{\pi}{2\beta}$
j	Depth of primary fixed point, *l* (mm)	$l = \frac{1}{\beta}\tan^{-1}\frac{1+\beta h_0}{\beta h_0}$	$l = \frac{\pi}{2\beta}$	$l = \frac{3\pi}{4\beta}$
k	Depth causing deflection angle zero *L* (mm)	$L = \frac{1}{\beta}\tan^{-1}\left[-(1+2\beta h_0)\right]$	$L = \frac{3\pi}{4\beta}$	$L = \frac{\pi}{\beta}$

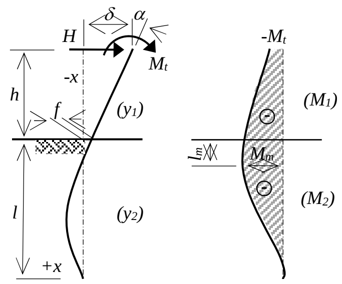
Table C.5.4.3.8-1b General Formula for Lateral Forces and Moments as External Forces (2) for Piles Protruding from the Ground

Differential equations of deflection curve		Portion above the ground: $EI\frac{d^4 y_1}{dx^4} = 0$ Portion in the ground: $EI\frac{d^4 y_2}{dx^4} + p = 0$ $p = k_H D y_2$ *H* : lateral force of a pile (N) M_t : moment as external force at the pile head (N-mm) *D* : pile diameter (mm) *E* : Young's modulus of the pile (N/mm^2) *I:* moment of inertia of the pile cross section (mm^4) k_H: coefficient of horizontal subgrade reaction (N/mm^3) *h:* height above the ground surface where *H* and M_t act (mm) $\beta = \sqrt[4]{k_H D / 4EI}$ (mm^{-1}) $h_0 = \frac{M_t}{H}$ (mm)

Pile condition		Pile Protruding from the Ground (h>0)		
Deflection curve and bending moment diagram		(a) Basic system 	(b) When $M_t = 0$ ($h_0 = 0$) 	(c) When pile head does not rotate 
a	Deflection curve, y (mm)	$y_1 = \frac{H}{6EI\beta^3}\left[\beta^3x^3 + 3\beta^3(h+h_0)x^2 - 3\{1+2\beta(h+h_0)\}\beta_x + 3\{1+\beta(h+h_0)\}\right]$ $y_2 = \frac{H}{2EI\beta^3}e^{-\beta x}\left[\{1+\beta(h+h_0)\}\cos\beta_x - \beta(h+h_0)\sin\beta_x\right]$	$y_1 = \frac{H}{6EI\beta^3}\{\beta^3x^3 + 3\beta^3hx^2 - 3(1+2\beta h)\beta_x + 3(1+\beta h)\}$ $y_2 = \frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h)\cos\beta_x - \beta h\sin\beta_x\}$	$y_1 = \frac{H}{12EI\beta^3}\left[2\beta^3x^3 - 3(1-\beta h)\beta^2x^2 - 6\beta^2hx + 3(1+\beta h)\right]$ $y_2 = \frac{H}{4EI\beta^3}e^{-\beta x}\left[(1+\beta h)\cos\beta_x + (1-\beta h)\sin\beta_x\right]$
b	Displacement of pile head, δ (mm)	$\delta = \frac{(1+\beta h)^3 + 1/2}{3EI\beta^3}H + \frac{(1+\beta h)^2}{2EI\beta^3}M_t$	$\delta = \frac{(1+\beta h)^3 + 1/2}{3EI\beta^3}H$	$\delta = \frac{(1+\beta h)^3 + 2}{12EI\beta^3}H$
c	Displacement of ground surface, f(mm)	$f = \frac{1+\beta(h+h_0)}{2EI\beta^3}H$	$f = \frac{1+\beta h}{2EI\beta^3}H$	$f = \frac{1+\beta h}{4EI\beta^3}H$
d	Pile head inclination angle, α (rad)	$\alpha = \frac{(1+\beta h)^2}{2EI\beta^3}H + \frac{1+\beta h}{EI\beta}M_t$	$\alpha = \frac{(1+\beta h)^2}{2EI\beta^3}H$	$\alpha = 0$
e	Bending moment at each section of the pile, M (N-mm)	$M_1 = -H(x+h) - M_t = -H(x+h+h_0)$ $M_2 = -\frac{H}{\beta}e^{-\beta x}\left[\beta(h+h_0)\cos\beta_x + \{1+\beta(h+h_0)\}\sin\beta_x\right]$	$M_1 = -H(x+h)$ $M_2 = -\frac{H}{\beta}e^{-\beta x}\{\beta h\cos\beta_x + (1+\beta h)\sin\beta_x\}$	$M_1 = \frac{H}{2\beta}\left[-2\beta_x + (1-\beta h)\right]$ $M_2 = \frac{H}{2\beta}e^{-\beta x}\left[(1-\beta h)\cos\beta_x - (1+\beta h)\sin\beta_x\right]$
f	Shear force at each section of the pile, S (N)	$S_1 = -H$ $S_2 = -He^{-\beta x}\left[\cos\beta_x - \{1+2\beta \bullet (h+h_0)\}\sin\beta_x\right]$	$S_1 = -H$ $S_2 = -He^{-\beta x}\left[\cos\beta_x - (1+2\beta h)\sin\beta_x\right]$	$S_1 = -H$ $S_2 = -He^{-\beta x}(\cos\beta_x - \beta h\sin\beta_x)$
g	Bending moment at pile head, M_0 (N-mm)	$M_0 = -M_t = -Hh_0$	$M_0 = 0$	$M_0 = \frac{1+\beta h}{2\beta}H$
h	Bending moment at a point along embedded section, M_m (N-mm)	$M_m = -\frac{H}{2\beta}\sqrt{\{1+2\beta(h+h_0)\}^2+1} \bullet \exp(-\beta l_m)$	$M_m = -\frac{H}{2\beta}\sqrt{(1+2\beta h)^2+1} \bullet \exp(-\beta l_m)$	$M_m = -\frac{H}{2\beta}\sqrt{1+(\beta h)^2} \bullet \exp(-\beta l_m)$
i	l_m (mm)	$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta(h+h_0)}$	$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h}$	$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{\beta h}$

j	Depth of primary fixed point, l (mm)	$l=\frac{1}{\beta}\tan^{-1}\frac{1+\beta(h+h_0)}{\beta(h+h_0)}$	$l=\frac{1}{\beta}\tan^{-1}\frac{1+\beta h}{\beta h}$	$l=\frac{1}{\beta}\tan^{-1}\left(\frac{\beta h+1}{\beta h-1}\right)$
k	Depth causing deflection angle zero L (mm)	$L=\frac{1}{\beta}\tan^{-1}\left[-\{1+2\beta(h+h_0)\}\right]$	$L=\frac{1}{\beta}\tan^{-1}\{-(1+2\beta h)\}$	$L=\frac{1}{\beta}\tan^{-1}(-\beta h)$

Table C.5.4.3.8-2 Calculation Formulas for Piles with Finite Length

	Differential equations of deflection curve	Deflection curve and bending moment diagram
	Portion above the ground: $EI\frac{d^4y_1}{dx^4}=0$ Portion in the ground: $EI\frac{d^4y_2}{dx^4}+p=0$ $p=k_H Dy_2$	

Formula		Section Above the Ground	Section Embedded in the Ground
a	Deflection curve, y (mm)	$y_1=f-\frac{1}{2EI\beta^3}(-C_1-C_2+C_3-C_4)x+\frac{M_t+Hh}{2EI}x^2+\frac{H}{6EI}x^3$	$y_2=\frac{1}{2EI\beta^3}\left[e^{-\beta x}(C_1\cos\beta_x+C_2\sin\beta_x)+e^{-\beta x}(C_3\cos\beta_x+C_4\sin\beta_x)\right]$
b	Bending moment at each section of the pile, M (N-mm)	$M_1=-H(x+h)-M_t$	$M_2=\frac{1}{\beta}\left[e^{\beta x}(C_1\sin\beta_x-C_2\cos\beta_x)+e^{-\beta x}(-C_3\sin\beta_x+C_4\cos\beta_x)\right]$
c	Shear force at each section of the pile, S (N)	$S_1=-H$	$S_2=e^{\beta x}\left[(C_1-C_2)\cos\beta_x+(C_1+C_2)\sin\beta_x\right]+e^{-\beta x}\left[-(C_3+C_4)\cos\beta_x+(C_3-C_4)\sin\beta_x\right]$
d	Displacement of pile head, δ (mm)	$\delta=f+\alpha h-\frac{M_t}{2EI}h^2-\frac{H}{6EI}h^3$	
e	Displacement of ground surface, f(mm)	$f=\frac{1}{2EI\beta^3}(C_3+C_1)$	
f	Pile head inclination angle, α (rad)	$\alpha=\frac{1}{2EI\beta^3}(-C_1-C_2+C_3-C_4)+\frac{M_t}{EI}h+\frac{H}{2EI}h^2$	
g	l_m (mm)	$\beta l_m^{(1)}=\tan^{-1}\frac{(C_3+C_4)-(C_1-C_2)\exp\left(2\beta l_m^{(0)}\right)}{(C_3-C_4)+(C_1+C_2)\exp\left(2\beta l_m^{(0)}\right)}$ (successive approximation formula)	

	Free Pile Tip	Hinged Pile Tip	Fixed Pile Tip
C_1	$\frac{H^*}{\Delta}\left[(1-\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]-\frac{\beta M^*}{\Delta}\left[(\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$	$\frac{H^*}{\Delta}\left[-\cos 2\beta l e^{-2\beta l}-e^{-4\beta l}\right]-\frac{\beta M^*}{\Delta}\left[(\sin 2\beta l-\cos 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$	$\frac{H^*}{\Delta}\left[(1+\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]-\frac{\beta M^*}{\Delta}\left[(\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$

C_2	$\frac{H^*}{\Delta}\left[(1-\cos 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[(2-\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$	$\frac{H^*}{\Delta}\left[(\sin 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[(\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$	$\frac{H^*}{\Delta}\left[(1-\cos 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[(2+\cos 2\beta l-\sin 2\beta l)e^{-2\beta l}-e^{-4\beta l}\right]$
C_3	$\frac{H^*}{\Delta}\left[1-(1+\sin 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[1-(\cos 2\beta l-\sin 2\beta l)e^{-2\beta l}\right]$	$\frac{H^*}{\Delta}\left[1-\cos 2\beta l e^{-2\beta l}\right]$ $+\frac{\beta M^*}{\Delta}\left[1+(\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}\right]$	$\frac{H^*}{\Delta}\left[1+(1-\sin 2\beta l)e^{-2\beta l}\right]$ $+\frac{\beta M^*}{\Delta}\left[1-(\cos 2\beta l-\sin 2\beta l)e^{-2\beta l}\right]$
C_4	$\frac{H^*}{\Delta}\left[(1-\cos 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[1-(2-\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}\right]$	$-\frac{H^*}{\Delta}\left[\sin 2\beta l e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[1+(\cos 2\beta l-\sin 2\beta l)e^{-2\beta l}\right]$	$\frac{H^*}{\Delta}\left[(1+\cos 2\beta l)e^{-2\beta l}\right]$ $-\frac{\beta M^*}{\Delta}\left[1+(2+\cos 2\beta l+\sin 2\beta l)e^{-2\beta l}\right]$
Δ	$1-2(2-\cos 2\beta l)e^{-2\beta l}+e^{-4\beta l}$	$1-2\sin 2\beta l e^{-2\beta l}+e^{-4\beta l}$	$1+2(2+\cos 2\beta l)e^{-2\beta l}+e^{-4\beta l}$
$H^* = H, \quad M^* = M_t + Hh$			

5.5 LONGINTUDINAL RESTRAINERS OR UNSEATING PREVENTION DEVICE

(1) Restrainer in this Article shall be referred to as the "unseating prevention device". The design for longitudinal restrainers or unseating prevention device shall be in accordance with Article 7.3 of Section 7 "Unseating Prevention System" of these Specifications, taking into account the interaction with the seat length, the device to limit excessive displacement and the device to prevent structure from settling.

(2) Friction shall not be considered to be an effective restrainer.

(3) The design force shall be that specified in Article 7.3. (Restrainers shall be designed for a force not more than 1.5 times the dead load reaction ($1.5R_d$), with the dead load reaction of the larger of the two adjoining spans or parts of the structure taken, as specified in Article 7.3).

(4) If the restrainer is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack shall be allowed in the restrainer so that the restrainer does not start to act until the design displacement is exceeded (refer to Article 7.3 for the design allowance length).

(5) Where a restrainer is to be provided at columns or piers, the restrainer of each span may be attached to the column or pier rather than to interconnecting adjacent spans. The design force for the restrainer shall be referred to Article 7.3.

(6) In lieu of restrainers, STUs may be used and designed for either the elastic force calculated in Section 4 of these Specifications or the maximum force effects generated by inelastic hinging of the substructure as specified in Article 5.3.4 of this Section.

5.6 HOLD-DOWN DEVICES

(1) For Seismic Performance Zones 2, 3, and 4, hold-down devices shall be provided at supports and at hinges in continuous structures where the vertical seismic force due to the longitudinal seismic load

opposes and exceeds 50 percent, but is less than 100 percent, of the reaction due to permanent loads. In this case, the net uplift force for the design of the hold-down device shall be taken as ten percent (10%) of the reaction due to permanent loads that would be exerted if the span were simply supported.

(2) If the vertical seismic forces result in net uplift, the hold-down device shall be designed to resist the larger of either:

- 120 percent (120%) of the difference between the vertical seismic force and the reaction due to permanent loads, or
- Ten percent (10%) of the reaction due to permanent loads.

5.7 BEARING SUPPORT SYSTEM

5.7.1 General

(1) This Article describes the fundamental design requirements of the bearing support systems under the Extreme Event I Limit State for earthquake effects. These provisions shall be applied in addition to other applicable code requirements and shall consider the function of the unseating prevention system as specified in Section 7 of these Specifications.

Moreover, the bearing support systems shall also satisfy the design and detail requirements under each of the other limit states of the *AASHTO LRFD Bridge Design Specifications (2012 or later), Section 14 – Joints and Bearings*.

(2) The bearing support system shall be classified under two types:

2.1) Type A Bearing Support – This bearing support system (typically applied in the Philippine bridges) shall be designed to resist the horizontal and vertical forces under Level 1 earthquake ground motion (EGM) and shall jointly resist the horizontal and vertical forces due to Level 2 EGM with the device for limiting excessive displacement as specified in Article 7.5 of these Specifications.

2.2) Type B Bearing Support – This bearing support type shall be designed to resist the horizontal and vertical forces due to Level 2 EGM. The bearing support system such as seismic isolation bearing and the elastic bearing that distributes the horizontal forces to the substructures shall conform to this provision.

(3) Both Type A and Type B bearing supports systems shall be designed with the provisions given on Article 5.7.3 and the design seismic forces specified in Article 5.7.2 and 5.3 of this Section.

(4) The bearing support system shall be designed considering the applicable requirements of the structural details of the support specified in Article 5.7.4 of these Specifications and Section 14 of the *AASHTO LRFD Bridge Design Specifications* in order to fully ensure functions of bearing support system. Bearings shall be consistent with the intended seismic response of the whole bridge.

(5) Unless otherwise noted, the resistance factor for bearings, ϕ, under Level 2 earthquake ground motion shall be 1.0.

Commentary C5.7.1

(1) These provisions for bearing support systems basically covers the requirements for extreme event limit state under earthquake effects. However, the support for other limit states under the *AASHTO LRFD Bridge Design Specifications* shall be complied with specially the design movements under other load conditions including temperature changes and live load effects.

Bearings have significant effect on the overall seismic response of the bridge. They provide the seismic load transfer link between a stiff and massive superstructure and a stiff and massive substructure. As a result, very high load concentrations can occur in the bearing components. The primary functions of the bearings are to resist the vertical loads due to dead load and live load and to allow for superstructure movements due to live load and temperature changes. Allowance for translation is made by means of rollers, shear deformation of elastomer or provision of sliding surface. Allowance for rotation is made by hinges, confined or unconfined elastomers, or spherical sliding surfaces. Resistance to translation is provided by bearing components or additional restraining elements or devices.

Historically, bearings have been very susceptible to seismic loads. Unequal loading during seismic events and much higher loads than anticipated have caused various types and levels of bearing damage. To allow movements, bearing often contain elements vulnerable to high loads and impact. The performance of bearings during past earthquakes needs to be evaluated in context with the overall performance of the bridge and the performance of the superstructure and substructure elements connected to the bearings. Rigid bearings have been associated with damage to the end cross-frames and the supporting pier or abutment concrete. In some cases, bearing damage and slippage has prevented more extensive damage.

(2) The bearing support serves as the connection between the superstructure and substructure. In this regard, it is necessary to allow the bearing support to be able to transmit the loads generated from the superstructure to the substructure effectively and to absorb the relative displacement between these components which correspond to the expansion or rotation of the superstructure caused by the live loads and changes in temperatures. The Philippine bridges under the DPWH jurisdiction are typically designed with bearings similar to Type A system using elastomeric bearing pads and a device to limit excessive displacements such as dowel bars or other means to fix the superstructure with the substructure.

The bearing support referred to here, consists of the bearing components installed at the connection point between the superstructure and the substructure. In order to ensure the above functions, the fastening members such as anchor bolts or set bolts (including the bearing seat mortar) to connect the bearing body with the superstructure and substructure shall be proportioned to appropriately resist the design forces.

The bearing support shall be capable of transmitting the seismic inertial force generated from Level 1 and Level 2 Earthquake Ground Motions that is acting on the superstructure to the substructure effectively. Especially, for case of a seismic-isolated bridge or a bridge with a support system capable of distributing the horizontal forces under earthquake, the bearing support system shall possess the functions mentioned above in order to prevent the whole bridge system from behaving differently from what has been assumed in the design. For bridges whose unseating prevention system needs a careful examination as described in Article 7.1 of these Specifications, Type B bearing supports shall be adopted since damages of the bearing supports may directly lead to bridge unseating.

As mentioned above, it is expected that the bearing support system should have several functions including transmitting loads or allowing resulting movements or displacements. A conventional bearing support system may have these functions concentrated and assured in a single structural component. However, it is not necessarily required for bearing support to have multiple functions concentrated in one structural component. For instance, individual bearing supports with single function can be combined together or requirements for multiple functions can be divided among different structural components. When one structural component is utilized to maintain multiple functions, the structural design will generally become complicated. It is also possible that loss of a certain function due to local damages or decrease in durability may have an impact on the other functions. In such a case, a combined bearing support system made up of several structural components with single functions or a separate structure in which the functions of a bearing support are shared by multiple structural components may be adopted depending on the structure type and scale of the bridge.

In some cases, it is better to allow the bearing support to have weak capacity so as to minimize the damages to the whole bridge system, when an excessively large seismic force acts from the superstructure to the substructure if the bearing support is designed to be too strong. This is so-called fuse theory on bearing support. However, as observed from the damages caused by the 1995 Hyogo-ken Nambu Earthquake, there were many cases found where the bearing supports were damaged together with substantial damages in the superstructure and substructures as well. Therefore, it is essential that the bearing support be regarded as one of the major structural members of a bridge and shall be designed to be capable of transmitting the superstructure inertial force to the substructure effectively.

The bearing support system shall be basically designed into a structure capable of resisting the superstructure inertial force in order to achieve the above design goal. However, in case that significant vibrations are not likely to occur due to the restraints of the abutment, or in an unavoidable condition due to structural constraints of the bearing support, it is acceptable that the bearing support is designed to work together with a structure limiting excessive displacement specified in Article 7.5 of these Specifications to supplement each other when resisting inertial forces. Type A bearing supports can then be selected to operate together with the structure limiting excessive displacement to resist the inertia forces for bridges where vibration does not occur easily during an earthquake like bridges with girders shorter than 50m and supported at both ends by an abutment. In case that the abutment is relatively flexible with low stiffness, i.e. so-called pier type abutment, Type B bearing supports shall be adopted. However, when Type A bearing supports are selected due to structural limitations of the bearing supports, careful examination is required on the seismic behavior of the bridge so that an appropriate unseating prevention system is designed.

Fig. C5.7.1-1 illustrates the above general concerns in the selection of the bearing supports.

With regards to detailed design and design methods for various kinds of bearing supports, other references can be used such as the *Handbook of Bearing Supports for Highway Bridges (Japan Road Association).*

Reference is also made to the *AASHTO Specifications* which categorize bearings based on the horizontal stiffness as follows:

- Rigid bearings that transmit seismic loads without any movement or deformations;
- Deformable bearings that transmit seismic loads limited by plastic deformations or restricted slippage of bearing components;
- Seismic isolation type bearings that transmit reduced seismic loads, limited by energy dissipation; and,
- Structural fuses that are designed to fail at a prescribed load.

For the deformable-type bearing, limited and repairable bearing damage and displacement may be allowed for the design earthquake. When both the superstructure and the substructure components adjacent to the bearing are very stiff, a deformable-type bearing can be considered. Elastomeric bearings have been seen to demonstrate reduced force transmission to the substructure. A bearing may also be designed to act as a "structural fuse" that will fail at a predetermined load changing the articulation of the structure, possibly changing its period and hence seismic response, and probably resulting in increased movements. However, this will require full consideration of forces and movements and of bearing repair/replacement details. It also requires the designer to address the inherent difficulty of detailing a structural element to fail reliably at predetermined load.

(3) The design seismic force for the static design method is provided for the design of Type A and Type B bearing supports under Article 5.7.2 of this Section while the design method of bearing supports are described in Article 5.7.3. However, when the design of bearing supports is carried out using the dynamic method, the maximum response value obtained from the dynamic analysis shall be used as the design seismic force disregarding the provisions of Article 5.7.2. The design method described in Article 5.7.3 shall be followed as well in this case.

The structural details for the bearing body, the joint section and the superstructure-substructure connection to the bearing supports shall be provided in order to allow the bearing support to fully perform its function (refer to Section 5.7.4 for details).

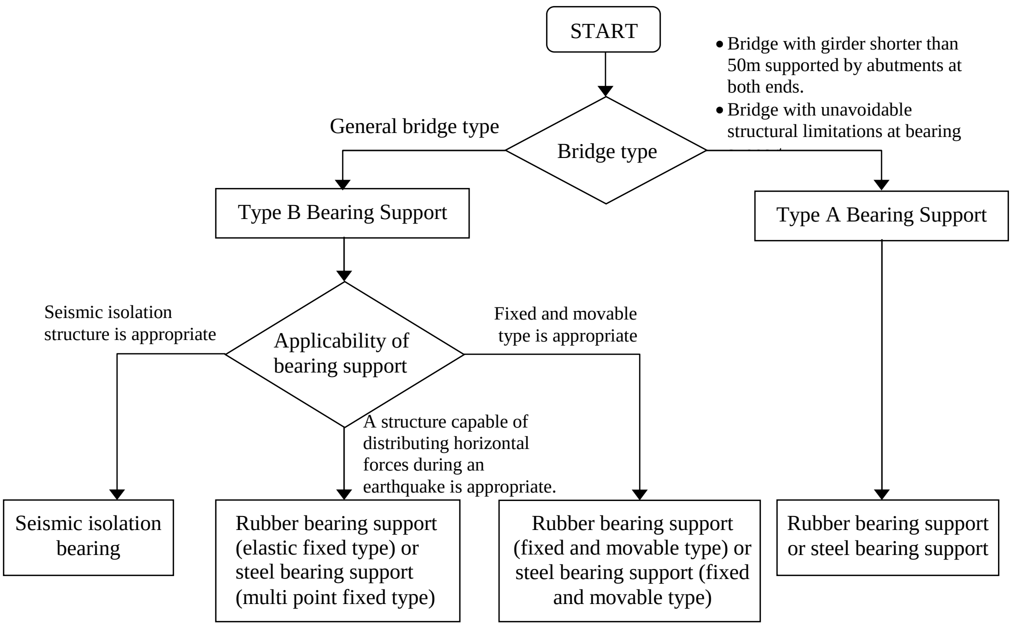


Figure C5.7.1-1 General Considerations for Selection of Bearing Supports.

5.7.2 Design Seismic Force for Bearing Support System

(1) The design horizontal seismic forces under Level 2 Earthquake Ground Motion (EGM) for bearing supports for Seismic Performance Zones 3 and 4 shall be equal to the lateral strength of columns or the plastic shear force considering plastic behavior of the column or pier. These forces under Level 2 EGM shall be fully resisted by Type B bearings supports but shall be resisted jointly by Type A bearing supports and the device for limiting excessive displacements. However, when the forces resulting from inelastic hinging analysis of the pier exceeds that of the elastic forces, the elastic forces may be used for the design of the bearing. For piers, the design forces shall comply with Article 5.3.4.3.e.

For Seismic Performance Zone 2, the design horizontal seismic force shall be as specified in Article 5.3.3 of these Specifications while Seismic Performance Zone 1, the design horizontal seismic force shall be as specified in Article 5.3.2 of these Specifications.

(2) The design horizontal seismic force for Level 1 Earthquake Ground Motion shall correspond to the inertial force calculated by using the design horizontal seismic coefficients (C_{sm}) specified in Article 3.6 of these Specifications multiplied by the dead load and factored live load reactions (with live load factor given in Article 1.5) when Type A bearing supports are used. No verification shall be needed for Type B bearing supports under Level 1 EGM. However, C_{sm} shall not be greater than 0.20, 0.25 and 0.30 for Ground Types I, II and III respectively.

(3) The design vertical seismic forces of both Type A and Type B bearing supports shall be obtained from Equations 5.7.2-1 and 5.7.2-2. However, the R_U value shall be taken as -0.3R_D when the R_U value obtained from Equation 5.7.2-2 does not exceed -0.3R_D for Type B bearing supports. Here, both downward seismic forces and downward reaction forces used in designing the bearing supports system shall be positive.

$$R_L = R_D + R_{LL} + \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \quad \text{(5.7.2-1)}$$

$$R_U = R_D - \sqrt{R_{HEQ}^2 + R_{VEQ}^2} \quad \text{(5.7.2-2)}$$

where:

R_L = Downward seismic force used for seismic design of bearing support system, (kN).

R_U = Upward seismic force used for seismic design of bearing support system, (kN).

R_D = Reaction force generated in the bearing support by the dead load of the superstructure, (kN).

R_{LL} = Reaction force generated in the bearing support by the vehicular live load using the live load factor specified in Article 1.5, (kN).

R_{HEQ} = Vertical reaction force generated in the bearing support system when the action of the horizontal force stipulated in Items (1) and (2) above is applied on the bearing support in the transverse direction to the bridge axis, (kN).

R_{VEQ} = Vertical seismic force generated by the design vertical seismic coefficient k_v which is obtained from the following Equations 5.7.2-3, (kN).

$$R_{VEQ} = \pm k_v R_D \quad \text{(5.7.2-3)}$$

k_v = Design vertical seismic coefficient. For Type B bearing supports, k_v shall be the product of the design horizontal seismic coefficient on the ground surface (A_s) specified in Article 3.6 of these Specifications for Level 2 EGM and the coefficient given in Table 5.7.2-1. For Type A bearing supports, k_v shall be the product of the design horizontal seismic coefficient A_s specified in Article 3.6 of these Specifications for Level 1 EGM and the multiplying coefficients given in Table 5.7.2-1.

Table 5.7.2-1 Multiplying Coefficient (k_v can be obtained by multiplying the Coefficients by the Design Horizontal Seismic Coefficients)

	Level 1 Earthquake Ground Motion (EGM) for Type A Bearing Supports	Level 2 Earthquake Ground Motion (EGM) for Type B Bearing Supports
Multiplying Coefficients	0.5	0.67

Commentary C5.7.2

(1) For Type B bearing support, it is necessary to consider that the assumed functions of the bearing support shall be ensured with regard to the seismic force generated by Level 2 Earthquake Ground Motion. The design horizontal seismic force that should be taken into account during design of a Type B bearing support is provided here. Moreover, the design for Type B bearing support shall be carried out for Level 2 Earthquake Ground Motion only. The bearing support design for Level 1 Earthquake Ground Motion will be satisfied when the design satisfies Level 2 Earthquake Ground Motion. In this regard, the design horizontal seismic force of the bearing support to be used in design with regards to Level 1 Earthquake Ground Motion is not stipulated in this Article.

The design horizontal seismic force for Level 2 Earthquake Ground Motion for Seismic performance Zones 3 and 4 shall be regarded as the horizontal force equivalent to the ultimate horizontal strength of a column or that corresponding to plastic shear when the plastic behavior of the column or pier is considered. However, the design seismic force stipulated here is meant for the bearing support on the condition that plasticity is assumed in the column. In case that plasticity is not considered for any of the structural members within a design vibration unit, or when considering the direction perpendicular to the bridge axis of a wall-typed pier or an abutment having a surplus seismic horizontal strength, there are cases in which the design seismic force for the bearing support becomes too large resulting to an unreasonable design. Nevertheless, different design conditions shall be given to the individual columns when the ultimate horizontal force of each column is used for the design of the bearing supports as in the case of a continuous bridge. Since this can cause difficulties in design practice, it is desirable that during the design of the bearing support, the horizontal force acting on each bearing support when subjected to Level 2 Earthquake Ground Motion shall be considered separately for each design vibration unit. When forces due to column/pier lateral strength or plastic hinging results in an unreasonable design, an alternative design shall be to use the elastic design forces derived from the load combination for Extreme Event I (earthquake loading) with *R-factor* being 1.0. Reference shall also be made to Article 5.3.4.3.e for pier design forces.

In case that seismic isolation bearings or rubber bearings which is capable of distributing the horizontal forces during an earthquake are adopted for the bridge, seismic performance design shall be carried out for Level 2 Earthquake Ground Motion by using the dynamic method. When designing the bearing supports, the basic sizes can be determined by using the reaction force of the bearing support obtained by applying statically the horizontal force equivalent to the ultimate horizontal strength of the column or the plastic shear forces corresponding to plastic hinging of the columns. However, in this case, it is understood that due to the difference between the vibration phases of the superstructure and the column, it is possible that the reaction force of the bearing support obtained by applying statically the horizontal force equivalent to the ultimate horizontal strength of the column or the plastic shear forces corresponding to plastic hinging of the columns will turn out to be smaller than the actual horizontal load during the evaluation. For this reason, the horizontal force multiplied by the dynamic modification factor c_m used in the calculation of the design displacement of a seismic isolation bearing specified in Article 8.3.3 of these Specifications is recommended to be adopted based on a comparison of the results of calculations for both static and dynamic analyses when a horizontal force is applied statically on the bearing support. When the seismic performance of a bridge with seismic isolation bearing or rubber bearings capable of distributing the horizontal forces during an earthquake is verified using the dynamic analysis method, the dynamic modification factor can be neglected since such effect is already included in the dynamic analysis. Nevertheless, even in the dynamic analysis, it is impossible to consider the effect of such difference in the vibration phases in the analysis when the response spectrum method with an equivalent linear method is to be adopted. It is suggested that the dynamic modification factor is taken into consideration here.

For Seismic Performance Zones 1 and 2, the design horizontal seismic force for Level 2 earthquake shall correspond to that of Articles 5.3.2 and 5.3.3 respectively.

(2) The design horizontal seismic force that shall be considered during the design of Type A bearing support is indicated here. Since the bearing support system should be capable of resisting the seismic force without loss of functions for Level 1 Earthquake Ground Motion which is comparatively high in occurrence frequency, it is stipulated that the horizontal force equivalent to the inertial force obtained by using the design horizontal seismic coefficient specified in this Article shall be regarded as the design horizontal seismic force. The seismic force to be applied for each bearing support shall be calculated by taking into account the distribution of forces within the design vibration unit. Since Type A bearing support may suffer from damages when the seismic force exceeds the design level, a structure limiting excessive displacement specified in Article 7.5 of these Specifications is required to be provided in order to prevent large relative displacement from occurring between the superstructure and the substructure.

(3) The design vertical seismic force that shall be considered during design of bearing support is provided here. Seismic forces in both horizontal and vertical directions acting on the bearing support shall be considered at the same time.

As shown in Figure C5.7.2-1, the reaction force R_{HEQ} in vertical direction will result even when a horizontal seismic force acts in the direction of the bearing support line. Therefore, this effect of the vertical force caused by the horizontal seismic force shall also be considered in addition to the seismic force in vertical direction R_{VEQ}. The possibility that the reaction force R_{VEQ} in vertical direction of the bearing support resulting from the seismic force in vertical direction and the vertical reaction force R_{HEQ} due caused by seismic force in horizontal direction will reach a maximum value simultaneously remains small. Therefore, the square root of the square of the sum of these two values shall be adopted.

R_{HEQ} can generally be calculated from Equation C5.7.2-1. Although R_{HEQ} changes according to the position of the bearing supports on one bearing support line can be used as R_{HEQ} value.

$$\left.\begin{aligned} H_B\, h_s &= \sum (R_{HEQi}\, x_i) \\ \sum R_{HEQi} &= 0 \\ R_{HEQi} &= K\,(x_i - x_o) \end{aligned}\right\} \quad \text{(C5.7.2-1)}$$

where:

R_{HEQi} = Reaction force generated in the *i*-th bearing support when the design horizontal seismic forces acts in the transverse direction to the bridge axis, (kN).

H_B = Design horizontal seismic force of bearing support specified in (1) and (2), (kN).

h_s = Vertical distance from the bearing seat surface to the center of gravity of the superstructure, (m). The maximum value of h_s on the bearing support line shall be used when there is a level difference in the seat surfaces on one bearing support line.

x_i = Horizontal distance from the gravity center of the superstructure to the *i*-th bearing support, (m). Both positive and negative values shall be considered.

K = A coefficient representing a proportional relationship. It can be obtained from Equation C5.7.2-1.

x_o = Distance from the balance point of R_{HEQi} to the center of gravity, (m). However, it becomes 0 when the center of gravity is in the center of the symmetrical section in the direction perpendicular to the bridge axis.

As shown in Figure C5.7.2-1, the R_{HEQi} value in Equation C5.7.2-1 shall be given Equation C5.7.2-2 when the section including the bearing support line is symmetrical. Among the values obtained, the maximum absolute value shall be R_{HEQi}. Also, as shown in Figure C5.7.2-1, in case that the bearing supports are arranged symmetrically in the direction perpendicular to the bridge axis and consisting of three bearing supports, the value of R_{HEQi} shall be given by Equation C5.7.2-3.

$$R_{HEQi} = \frac{H_B h_s}{\sum x_i^2} x_i \quad \text{(C5.7.2-2)}$$

$$R_{HEQ} = \frac{H_B h_s}{2x_1} \quad \text{(C5.7.2-3)}$$

When designing for Level 2 Earthquake Ground Motion, the design vertical seismic coefficient shall be the product of the design horizontal seismic coefficient on the ground surface specified in this Article multiplied by the coefficient shown in Table 5.7.2-1. Here, the coefficient is set to 0.67 in case of Type II earthquake ground motion. The reason for this is that the possibility of a strong earthquake ground motion in the vertical direction is considered to exist in the areas near the hypocentral regions as in the case of Kobe area during the 1995 Hyogo-ken Nambu Earthquake.

The lower limit of $-0.3R_D$ has been set for the upward seismic force for Type B bearing support. The purpose of this is to fully ensure safety against the earthquake ground motions in the vertical direction in the design of Type B bearing support. However, it is acceptable that only the seismic force in vertical direction is considered when the value of $-0.3R_D$ is adopted for the upward design vertical seismic force of the bearing support. The seismic force in horizontal direction may be not be taken into account at the same time. In case that the upward seismic force in the vertical direction is not acting on the bearing support, that is to say, it is acceptable that the value of $-0.3R_D$ can be neglected as the design vertical seismic force when the design vertical seismic force R_U in Equation 5.7.2-2 is positive and when a bearing structure which does not restrict the vertical displacement is adopted. In this case, however, a structure which can guarantee the functions of a bearing support even when the relative displacement in the vertical direction occurs, shall be selected. In case that a structure restricting vertical displacement is designed as a measure to counter the upward force, the lower limit for the upward seismic force shall be taken as $-0.1R_D$ when Type A bearing support is chosen.

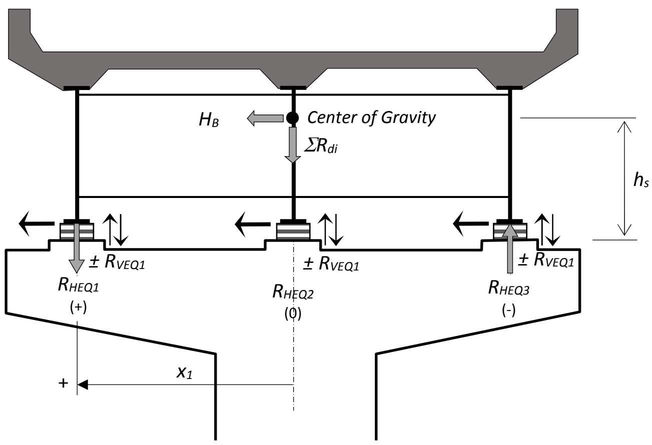


Figure C5.7.2-1 Vertical Reaction Force R_{HEQ} Generated in the Bearing Support due to Horizontal Seismic Force & Vertical Force R_{VEQ} Generated in the Bearing Support due to Vertical Seismic Force

5.7.3 Design of Bearing Support System

(1) Design of Type B bearing supports.

For Type B bearing supports, the sectional forces generated in the bearing support and the attached members shall not exceed the nominal (ultimate) strength when the action of the design horizontal seismic forces specified in Article 5.7.2(1) of this Section and the design vertical seismic forces in Article 5.7.2(3) are applied.

Furthermore, for Type B rubber bearing support or seismic isolation bearings, the shear strain generated in the bearing support shall not exceed the allowable value, and the safety of the bearing support against buckling shall be verified in addition to the above provisions. Here, the allowable shear strain shall be properly determined according to the characteristics of rubber bearings or seismic isolation bearings actually used, in order to protect them from shear failure. The stiffness of the bearing support shall be kept with ±10% of the design value.

(2) Design of Type A bearing supports

For Type A bearing supports, the sectional forces generated in the bearing support and the attached members shall not exceed the nominal (ultimate) strength when the action of the design horizontal seismic forces specified in Article 5.7.2(2) and the design vertical forces in Article 5.7.2(3) are applied.

Furthermore, for Type A rubber bearing supports, the shear strain generated in the bearing support shall not exceed the allowable value, and the safety of the bearing support shall be ensured in addition to the above provisions. Here, the allowable shear strain shall be properly determined according to the characteristics of the rubber bearing actually used, in order to protect the bearing from shear failure.

Commentary C5.7.3

(1) When Type B bearing supports are used, the ultimate strengths of the bearing support body and the attached members shall be calculated based on the nominal (ultimate) strength of the member. The purpose for this is to allow the bearing support to have the necessary strength to guarantee its function of transmitting the superstructure inertial force to the substructure effectively. For this reason, considering the seismic force generated due to the Level 2 Earthquake Ground Motion, the nominal (ultimate) strength of a bearing support is set to the yield limit. However, in case that calculation of the ultimate strength and deformation performance can be done based on careful investigation, the design can be carried out according to the results of such calculations.

In the bearing support system, the superstructure-substructure connections to the bearing support shall comply with the requirements of Articles 3.6 and 3.8 and Article 5.3.

The allowable shear strain of the body of a rubber bearing or an isolation bearing shall be determined properly in order to guarantee the necessary safety of the bearing body against shear failure. Generally, the design shear strain of a bearing body calculated by Equation C5.7.3-1 is kept below 250%.

This limit value is set taking into consideration that it is within the range which shows a fairly stable characteristics of the horizontal force-horizontal displacement relationship of a laminated rubber bearing (which is made of natural rubber, chloroprene rubber and high damping rubber) and that it is the value taking into account the allowance for the shear fracture of a rubber bearing or a seismic isolation bearing. In addition, the allowable shear strain of the rubber bearing or the seismic isolation bearing shall be determined based on careful investigations through experiments when certain types of rubber without any available data is to be adopted for rubber bearing or a seismic isolation bearing.

$$\gamma = \frac{u_B}{\sum_i^n t_{ei}} \qquad \text{(C5.7.3-1)}$$

where:

γ : Shear strain generated in a rubber bearing or a seismic isolation bearing.

u_B : Horizontal displacement generated in the bearing support by the design horizontal seismic force, (mm).

t_{ei} : Thickness of the *i*-th layer of the rubber, (mm).

n : Number of rubber layers.

In the previous version of the Seismic Design Specifications of JRA (1996), for the purpose of preventing local damages to a rubber bearing, it was stipulated that the local shear strain occurring in a rubber bearing due to the actions of the vertical loads and horizontal force upon it should be kept below the allowable value during design. However, revisions have been made based on the considerations that the conventional equation may not necessarily be accurate in the evaluation of the local failure. Moreover, during the design for shear strain of a bearing support within the range of vertical loads used generally, as mentioned above, it is also possible to examine the local shear failure at the same time. Furthermore, according to the results of fatigue tests for rubber bearings carried out recently, the fracture bearing occurs 300 – 500 times, 10,000 times and 100,000 times in case that the shear strain is assumed to be 300%, 200% and 100% respectively, when subjected to cyclic actions of shear deformation. The design of rubber bearing can vary greatly according to the different properties of bridges. Generally speaking, the possibility of the shear strain to cause fracture in the rubber bearing remains rather low even when a large shear strain assumed in seismic design occurs dozens of times due to the seismic response. Therefore, during design of rubber bearing, examinations of the local shear strain shall not be carried out this time. Nevertheless, for design under other limit states or under ordinary conditions, it becomes necessary to verify the local shear strain from the point of view of durability.

An increase in the height of a bearing support may cause buckling in the bearing support causing a possible overturn. Consequently, safety of a bearing support against buckling shall also be subject for examination. For design of buckling of the bearing body, the Handbook of Bearing Supports for Highway Bridges (Japan Road Association) is recommended for reference. In addition, it is suggested that the secondary shape factor is set to be more than 4 in order to maintain the horizontal force support function of a rubber bearing which is stable even when it is subjected to seismic deformation.

In case that the bearing structure capable of distributing horizontal forces during an earthquake with rubber bearing is adopted, the stiffness of the rubber bearing used in practice shall be limited within ±10% of the design value. The stiffness of a rubber bearing can have a big influence on the strengths of the inertia forces distributed from the superstructure to the substructure.

During the design of the bolt for fixing the upper steel plate to the girder or the connections with the bearing support under the superstructure inertial force, the rotation force that may occur in the bearing support caused by the height of the support shall be taken into account. When a shear key is adopted to transmit seismic force, the fixed bolts shall be designed to resist the force in vertical direction in order to enable the shear key to function reliably.

(2) For Type A bearing support, the sectional forces generated in the bearing body and the fastening members shall be kept below the nominal (ultimate) strength when it is subjected to seismic forces in both horizontal and vertical directions under the Level 1 Earthquake Ground Motion. Article 7.5.4 of this Section shall be adopted when considering the superstructure-substructure connections to the bearing support.

Since the bearing is required to be designed to function satisfactorily when subjected to Level 1 Earthquake Ground Motion, the shear strain of a bearing body calculated by Equation C5.7.3-1 shall be verified not to exceed the allowable shear strain when rubber bearings are used. The standard of 150% can be taken here, since there is a limit of 200% - 175% for the allowable shear strain for conventional type of rubber pad bearing. The allowable shear strain of the rubber bearing shall be determined appropriately based on careful studies through experiments, in case that a certain type of rubber without any available data is adopted for a rubber bearing.

5.7.4 Structural Details of Bearing Support System

(1) Type B bearing support shall be connected securely to the superstructure and substructure and be structurally functional.

(2) The superstructure and substructure connection with the bearing support shall be sufficiently reinforced to withstand seismic forces.

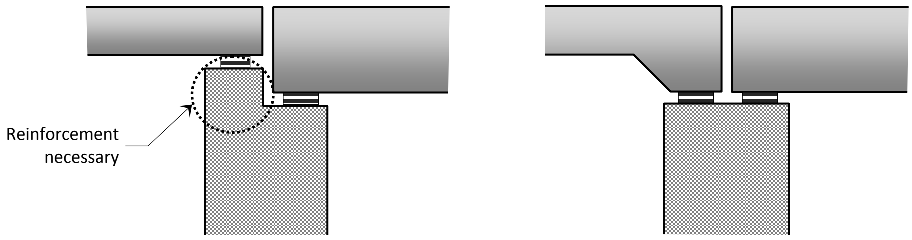
(3) Highly ductile materials shall be used for these parts and stress concentration shall be minimized in order to protect the bearing support from brittle failure.

Commentary C5.7.4

(1) When Type B bearing supports are used, a structure or device which can completely guarantee the functions of the bearing support should be adopted for the joint connection between the bearing body and the superstructure and substructure. Especially, for a seismic-isolated bridge or a bridge with a rubber bearing support system capable of distributing the horizontal forces during an earthquake, bolts shall be used for the connection of the upper steel plate and the superstructure in order to connect the bearing body with the superstructure and substructure firmly. For this joint section, as specified in Article 5.7.3(1), a structure capable of withstanding the seismic forces in both horizontal and vertical directions shall be chosen.

(2) The design on the safety of the superstructure-substructure connections with the bearing support shall be carried out by following the relevant specifications for Steel Structures and Concrete Structures as specified in the *AASHTO LRFD Bridge Design Specifications* (*2012 edition*), to resist the seismic forces assumed in the design of a bearing support effectively. Strengthening of the connection section for steel and concrete structures may be necessary. Reference shall also be made to *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (*2011*) for connection design.

In case adjacent girders have substantial difference in their heights, necessitating pier caps to be truncated as shown in Figure C5.7.4-1, the possibility of collision between the girder and the higher section of the pier cap may occur generating large seismic collision force. This may cause substantial damages to the girder or the lower section of the higher pier cap. In such case, as illustrated in Figure C5.7.4-1, effective measures shall be taken such as unifying the heights of the pier crown portions or reinforcing the higher portion of the pier crown and the girder end.



(a) Pier having a truncated section in pier cap

(b) Pier with cap being unified in height

Figure C5.7.4-1 Measures for Dealing with Adjacent Girders of Different Heights

(3) Since the possibility of an extremely large seismic force acting upon the bearing support exists, highly ductile materials shall be used for the bearing support in order to avoid brittle failure. Moreover, materials highly resistant to abrasion or rust of the moving/sliding section are recommended to be adopted to ensure proper function of the bearing support.

5.8 GAP BETWEEN ADJACENT GIRDERS AND SUBSTRUCTURES

(1) When designing the ends of the superstructure, the necessary gap between the ends of the two adjacent girders shall be secured to prevent collision between the two adjacent superstructures, the superstructure and the abutment, or the superstructure and the truncated section of the pier head, when subjected to both Level 1 and Level 2 Earthquake Ground Motions. In particular, for seismically-isolated bridges, sufficient spacing of adjacent girders shall be provided at the ends of the superstructures in order to ensure the expected behavior of seismic isolation. However, when designing ordinary bridge other than seismically-isolated bridges, the gap between the ends of two adjacent girders can be determined by considering that the collision do not occur for Level 1 Earthquake Ground Motion, if verification confirms that the collision will not affect the seismic performance of the bridge when subjected to Level 2 Earthquake Ground Motion.

(2) The gap between the ends of two adjacent girders to avoid the collision between adjacent superstructures, a superstructure and an abutment, or a superstructure and a truncated section of a pier head, shall not be less than the value obtained from Equation 5.8-1. However, for a complicated bridge in which the seismic behavior should be obtained by a dynamic analysis method, the relative displacement derived from the dynamic analysis shall be taken as u_s, in Equation 5.8-1.

$$S_B = \begin{cases} u_s + L_A \text{ (between a superstructure and an abutment, or a superstructure and a truncated section of a pier head)} \\ c_B\, u_s + L_A \text{ (between two adjacent girders)} \end{cases} \qquad (5.8\text{-}1)$$

where:

S_B : Gap/space between adjacent decks at the ends of superstructures shown in Figure 5.8-1, (mm).

u_s : Maximum relative displacement between a superstructure and a substructure, generated at the position for calculating the gap between adjacent decks when subjected to the Level 1 and Level 2 Earthquake Ground Motion, (mm).

L_A : Gap allowance for adjacent girders, (mm). Normally taken as 15mm.

c_B : Gap modification factor for the difference in natural period. The values in Table 5.8-1 are based on the natural period difference ΔT of the two adjacent girders.

Table 5.8-1 Gap Modification Factor on Natural Period Difference of Adjacent Girders

Ratio of Natural Period Difference between Adjacent Girders $\Delta T/T_1$	c_B
$0 \leq \Delta T/T_1 < 0.1$	1
$0.1 \leq \Delta T/T_1 < 0.8$	$\sqrt{2}$
$0.8 \leq \Delta T/T_1 \leq 1.0$	1

Note: Here, $\Delta T = T_1 - T_2$, and T_1 and T_2 represent the natural periods of two adjacent girders, respectively. However, T_1 is assumed equal to or greater than T_2.

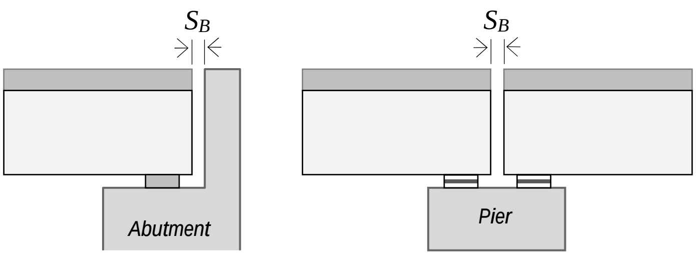


Figure 5.8-1 Gap between Ends of Superstructures

Commentary C5.8

(1) In case that a collision is likely to occur between two adjacent superstructures, a superstructure and an abutment, or a superstructure and the truncated portion of a pier crown, a gap shall be provided at the ends of the superstructures in the bridge axis direction so as to prevent any loss in the seismic performance of the bridge caused by the collision. Here, the basic requirement is provided for the gap between the adjacent decks in the bridge axis direction.

An appropriate width of the gap shall be decided considering Level 1 and 2 Earthquake Ground Motions. Particularly in case of a seismically-isolated bridge, the gap has to be designed to avoid collision between the ends of superstructures. This is because the purpose of a seismically-isolated bridge is to create a long natural period and improve the damping performance of the bridge through deformation of the isolation bearings. If such deformation in the isolation bearings fails to occur because the displacement of the superstructure has been restricted due to collision, the bridge will show some behavior that were not assumed in design including loss of the expected damping effect.

However, with regard to Level 2 Earthquake Ground Motion, if a big gap is kept at the ends of superstructures in order to avoid collision, the expansion joint is likely to be designed into a structure being too big to be economical and may cause problems in management, driving conditions, vibration and noise. For this reason, for the case of a bridge other than a seismically-isolated bridge, when confirmation is made to ensure that the collision do not impede the seismic performance of the bridge, it is acceptable that the gap can be designed to avoid collision with regards to Level 1 Earthquake Ground Motion.

If a collision occurs between two adjacent superstructures, a superstructure and an abutment, or a superstructure and the truncated portion of the pier crown, it is possible that the seismic performance of the whole bridge system may be improved because the displacement of the superstructure is restricted by such a collision. However, attention shall be given to avoid loss of the seismic performance, which may be caused by damages at the ends of superstructures, the abutment or the truncated portion of the pier crown after the collision. Therefore, when verification is carried out to consider the collision, it shall be ensured that an excessively strong force is not generated at the location of the collision and adoption of a structure that can reduce the collision effect by using a buffer at the location of possible collision shall be taken into account. During verification, it is recommended that while considerations are made in these structural aspects to prevent loss of seismic performance of the bridge, the seismic behaviors of the entire bridge system shall be properly evaluated through dynamic analysis. Such verification can be focused on the influence of the collision on the seismic performance of the whole bridge system. However, there are some other structures available for choice to guarantee the actual gap during a large-scale earthquake as in the case of a knock-off structure or an absorption system of big displacements. Use of these structures can be investigated according to necessity.

The gap between the ends of superstructures described above refers to the spacing in the bridge axis direction. Generally, if an expansion joint has suffered some damage, the displacement of the superstructure is not restricted in the perpendicular direction to the bridge axis. For this reason, gap is not required in the perpendicular direction to the bridge axis. However, in case that isolation design is adopted to expect seismic isolation effect in the perpendicular direction to the bridge axis, if there is no damage in the expansion joint or if some special expansion joint or excessive displacement stoppers are used to restrict the displacement of the superstructure in the perpendicular direction to the bridge axis even after the expansion joint has suffered some damage, a gap shall have to be considered for the members confining the displacement of the superstructure in the perpendicular direction to the bridge axis. However, in case that an excessive displacement stopper is adopted for the purpose of restricting the displacement of the superstructure in the perpendicular direction to the bridge axis, since no relative displacement will occur in the superstructure and substructure in this direction, expectation of seismic isolation effect shall not be allowed.

When two superstructures adjacent to each other are greatly different in weights, it can hardly be avoided that the heavier superstructure will push the lighter superstructure during collision between the two. In this case, special attention shall be given to the gap between these structures and at the same time, measures shall be taken to leave sufficient extra seat length for the girder as given in the unseating prevention system in Section 7 of these Specifications.

(2) In order to avoid collision between the two adjacent superstructures, a superstructure and an abutment or a superstructure and the truncated portion of the pier crown, it is stipulated that the necessary gap between superstructures shall be obtained as the sum of the relative displacement occurring at the ends of the superstructures and the gap allowance between the adjacent decks.

The relative displacement u_s occurring at the end of the superstructure is basically regarded as the maximum relative displacement between the superstructure and substructure that is obtained from the verification of seismic performance considering Level 2 Earthquake Ground Motion and can generally be calculated by the method described below:

1) When the supports are elastic rubber bearing type

In case that the supports are isolation bearings or seismic horizontal force distributing bearings etc., or the Type B rubber bearing specified in Article 5.7 of these Specifications, the relative displacement occurring between the superstructure and substructure at the supporting point when Level 2 Earthquake Ground Motion is acting on the design vibration unit which includes the supporting point concerned shall be taken as u_s. Generally, u_s can be regarded as the horizontal displacement occurring in the rubber bearing.

2) When the supports are moveable bearing type

When Level 2 Earthquake Ground Motion acts upon the design vibration units containing the superstructure and substructure respectively, the maximum relative displacement occurring at the supporting point shall be taken as u_s. However, the displacement of the design vibration unit consisting of an abutment can be regarded as zero because the deformation of an abutment is relatively small.

In Equation 5.8-1, the relative displacement between the superstructure and abutment is provided assuming that the abutment has sufficiently large stiffness and the relative displacement of two consecutive superstructures on a pier is given based on the relative displacement response spectrum. When the natural periods of the two consecutive superstructures are the same, vibrations occur with the same phase for the same seismic input motions. In this case, the relative displacement becomes zero in theory. On the contrary, when the natural periods of the two consecutive superstructures differ from each other, the two superstructures will vibrate separately. As a result, a relative displacement will occur between the two superstructures. Moreover, if the two natural periods are significantly different, the response displacement of the superstructure with a longer natural period will become more dominant than the response displacement of the superstructure with a shorter natural period, bringing the displacement to come closer to that of the superstructure with the longer natural period. The modification factor c_B on the difference between the natural periods of the gap between the adjacent decks shown in Table 5.8-1 has been determined by taking these vibration characteristics into consideration and by referring to the relative displacement spectrum calculated based on the strong earthquake records of horizontal components of the 63 elements which have been obtained from the earthquakes of magnitude 6.5 or greater recorded in Japan.

In case that a pier is supporting two consecutive superstructures, when calculating the gap allowance by using the modification factor on the difference between two natural periods, for the relative displacement, u_s, between the superstructure and substructure, the maximum relative displacement with the longer natural period (natural period T_1) shall be used. Since the relative displacement occurring between the superstructure and substructure changes in response to various conditions, during verification of seismic performance by using the dynamic analysis method, the relative displacement between the two structures, u_s, shall be determined based on the analysis results.

When calculating the maximum relative displacement between the superstructure and the substructure, it is not necessary to take the effect of the lateral movement of the ground into account. This is because lateral movement of the ground generally occurs during and after an earthquake. According to the characteristics of an earthquake ground motion, while the response of a superstructure is still large, the displacement of the ground caused by the lateral movement is considered to be small. It is also because in case that a collision happens due to the lateral movement of the ground, the speed of the collision is thought to be extremely slow.

The gap allowance is designed to deal with the margin of error in the installation of superstructures during construction. Generally, the standard of around 15mm shall be taken for this allowance.

SECTION 6:

EFFECTS OF SEISMICALLY UNSTABLE GROUND

6.1 GENERAL

(1) The effects of the unstable ground shall be taken into account in the verification of seismic performance of a bridge when the ground is expected to be in an unstable state during an earthquake. Unstable ground is defined as an extremely soft soil layer in seismic design, or a sandy layer affecting the bridge due to the liquefaction and lateral spreading.

(2) In addition to the verification of the seismic performance of a bridge with conditions indicated in Item (1) above, the case in which the ground is assumed to be stable shall also be considered in order to ensure the seismic performance of the bridge for both stable and unstable ground conditions.

Commentary C6.1

(1) Past cases of earthquake damage have revealed that a decline in the strength of an extremely soft clayey layer or silt layer during an earthquake, or liquefaction in a saturated sand layer which is accompanied by liquefaction-induced ground flow of the ground can seriously affect the seismic performance of a bridge. Therefore, it has been stipulated that such effect shall be taken into account during verification of seismic performance of a bridge when the ground becomes unstable during an earthquake.

For clayey layer and silt layer, there are cases in which the soil strength declines when the layer is subjected to repeated deformations during an earthquake. This tendency is recognized especially in layers having a high sensitivity ratio such as soft clayey layer or silt layer in the surface layer.

When liquefaction occurs, it is possible that structures with an apparently high specific gravity will subside while structures with an apparently low specific gravity will rise. Furthermore, the structures that are built to resist the earth pressure will be pushed forward due to the increase of the earth pressure. The structures, such as foundation, that are expected to have horizontal resistance may loose the resistance capacity and considerable displacement may occur. Furthermore, for inclined ground near the water front line, it is possible that liquefaction accompanied by a liquefaction-induced ground flow may occur. For this reason, it has been stipulated that these effects have to be taken into account in the verification of seismic performance as well.

(2) As the ground changes into an unstable state, the response properties of a structure tend to become complicated. Even if a certain ground has been estimated to be unstable, it is possible that it may not be in the same conditions as it has been assumed in the original design due to the physical properties of the earthquake ground motion or the ground itself. Therefore, the design condition in which no instability of the ground is expected shall also be a subject for verification of seismic performance. From the verification results under both conditions, the more disadvantageous condition shall be chosen and adopted for design.

That is to say, in case that it has been judged that liquefaction affecting the bridge will possibly occur, the condition in which no liquefaction will happen shall also be examined. Then, the result having more severe conditions shall be used for seismic design. In addition, in case that it has been determined that liquefaction-induced ground flow affecting the bridge is likely to occur, the following three cases shall be examined during verification of the bridge seismic performance:

1) when it is assumed that liquefaction-induced lateral spreading will also occur,
2) when it is assumed that only liquefaction will occur, and
3) when it is assumed that neither liquefaction nor liquefaction-induced lateral spreading will occur.

Based on the results, the case that has the most severe effect on the bridge shall be selected for seismic design of the bridge.

6.2 GEOTECHNICAL PARAMETERS OF EXTREMELY SOFT LAYER AND SANDY LAYER PRONE TO LIQUEFACTION

6.2.1 General

For an extremely soft soil layer in seismic design specified in Article 6.2.2 or a sandy layer specified in Article 6.2.3 which may affect the bridge due to liquefaction, the geotechnical parameters used in the seismic design shall be reduced in accordance with the provisions in Article 6.2.4.

Commentary C6.2.1

When liquefaction is estimated to occur within an extremely soft clayey layer or a silt layer, there is a possibility for the strength and bearing capacity of the soil to decline. In view of this condition, it is stipulated that the geotechnical parameter used for seismic design shall be either zero or a reduced value according to Article 6.2.4. ***However, when obtaining the design horizontal seismic coefficient, the natural period shall be calculated without considering the reduction of the geotechnical parameter as designated in Article 6.2.4.***

6.2.2 Assessment of Extremely Soft Soil Layer in Seismic Design

For a clayey layer or a silt layer located within three meters from the ground surface, and having a compressive strength of 20 kPa (kN/m^2) or less obtained from an unconfined compression test or an in- situ test, the layer shall be regarded as an extremely soft layer in seismic design.

Commentary C6.2.2

Generally, when the clayey soil or silt soil has a compressive strength of 20 kPa (kN/m^2) or less, the test sample of such soil will be so soft that it can hardly stand by itself during the test. Because of this, the effective bearing capacity of the foundation during an earthquake from this kind of soil cannot be expected. The soil layer of this case shall therefore be regarded as extremely soft in seismic design.

6.2.3 Assessment of Soil Liquefaction

(1) Sandy Layer Requiring Liquefaction Assessment

For an alluvial sandy layer having all of the following three conditions, liquefaction assessment shall be conducted in accordance with the provisions specified in Article 6.2.3(2) below, since liquefaction may affect the bridge performance during an earthquake.

1) Saturated soil layer with depth less than 20 m below the ground surface and having ground water level higher than 10 m below the ground surface.

2) Soil layer containing a fine content (*FC*) of 35% or less, or soil layer having plasticity index, I_P, less than 15, even if *FC* is larger than 35%.

3) Soil layer having a mean particle size (D_{50}) of less than 10 mm and a particle size at 10% passing (D_{10}) (on the grading curve) is less than 1 mm.

(2) Assessment of Liquefaction

For the soil layer requiring liquefaction assessment according to the provisions specified in Item (1) above, the liquefaction resistance factor F_L, shall be calculated by Equation 6.2.3-1. When the result turns out to be less than 1.0, the layer shall be regarded as a soil layer having liquefaction potential.

$$F_L = R / L \qquad (6.2.3\text{-}1)$$

$$R = c_w R_L \qquad (6.2.3\text{-}2)$$

$$L = r_d\, k_{hgL}\, \sigma_v / \sigma'_v \qquad (6.2.3\text{-}3)$$

$$r_d = 1.0 - 0.015x \qquad (6.2.3\text{-}4)$$

$$k_{hgL} = F_{pga}\, PGA \qquad (6.2.3\text{-}5)$$

$$\sigma_v = \gamma_{t1} h_w + \gamma_{t2}(x - h_w) \qquad (6.2.3\text{-}6)$$

$$\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2}(x - h_w) \qquad (6.2.3\text{-}7)$$

$$c_w = \begin{cases} 1.0 & (R_L \leq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases} \qquad (6.2.3\text{-}8)$$

where:

F_L : Liquefaction resistance factor.

R : Dynamic shear strength ratio.

L : Seismic shear stress ratio.

c_w : Modification factor on earthquake ground motion.

R_L : Cyclic triaxial shear stress ratio to be obtained from Equation 6.2.3-9 in Item (3) below.

r_d : Reduction factor of seismic shear stress ratio, in terms of depth.

k_{hgL} : Design horizontal seismic coefficient at the ground surface for Level 2 EGM.

F_{pga} : Site coefficient for peak ground acceleration specified in Article 3.5.3.

PGA : Peak ground acceleration coefficient on rock, as given in Article 3.6.

σ_v : Total overburden pressure, (kN/m^2).

σ'_v : Effective overburden pressure, (kN/m^2).

x : Depth from the ground surface, (m).

γ_{t1} : Unit weight of soil above the ground water level, (kN/m^3).

γ_{t2} : Unit weight of soil below the ground water level, (kN/m^3).

γ'_{t2} : Effective unit weight of soil below the ground water level, (kN/m^3).

h_w : Depth of the ground water level, (m).

(3) Cyclic triaxial shear stress ratio

Cyclic triaxial shear stress ratio R_L shall be calculated by Equation 6.2.3-9.

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6x10^{-6} \bullet (N_a - 14)^{4.5} & (14 \leq N_a) \end{cases} \quad \text{.............................. (6.2.3-9)}$$

where:

(For Sandy Soil)

$$N_a = c_1 N_1 + c_2 \quad \text{.......... (6.2.3-10)}$$

$$N_1 = 170N / (\sigma'_v + 70) \quad \text{.......... (6.2.3-11)}$$

$$c_1 = \begin{cases} 1.0 & (0\% \leq FC < 10\%) \\ (FC + 40) / 50 & (10\% \leq FC < 60\%) \\ FC / 20 - 1 & (60\% \leq FC) \end{cases} \quad \text{.......... (6.2.3-12)}$$

$$c_2 = \begin{cases} 0 & (0\% \leq FC < 10\%) \\ (FC - 10) / 18 & (10\% \leq FC) \end{cases} \quad \text{.......... (6.2.3-13)}$$

(For Gravelly Soil)

$$N_a = \{1 - 0.36 \log_{10}(D_{50}/2)\}\ N_1 \quad \text{.......... (6.2.3-14)}$$

R_L : Cyclic triaxial shear stress ratio.

N : N-value obtained from the standard penetration test.

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2.

N_a : Modified N value taking into account the effects of grain size.

c_1, c_2 : Modification factors of N value on fine content.

FC : Fine content, (%) (percentage by mass of fine soil passing through the 75μm mesh).

D_{50} : Mean grain diameter, (mm).

Commentary C6.2.3

These standards for the assessment of liquefaction have been established by adding the analysis on the examples of the Hyogo-ken Nanbu Earthquake (Japan) in 1995 to the research results accumulated since the Niigata Earthquake (Japan) in 1964.

(1) Almost all past cases of liquefaction during an earthquake had occurred in the alluvial sandy layers. However, during the Hyogo-ken Nanbu Earthquake and other recent earthquakes, liquefaction occurred in soil layers other than the alluvial sandy layers. Therefore, the range of soil layers requiring the assessment of liquefaction will be as follows (refer to Figure C6.2.3-1):

1) The depth of the soil layer for assessment has been set to within 20 m below the ground surface based on the studies of the past experiences and the seriousness of the soil effects on structures.

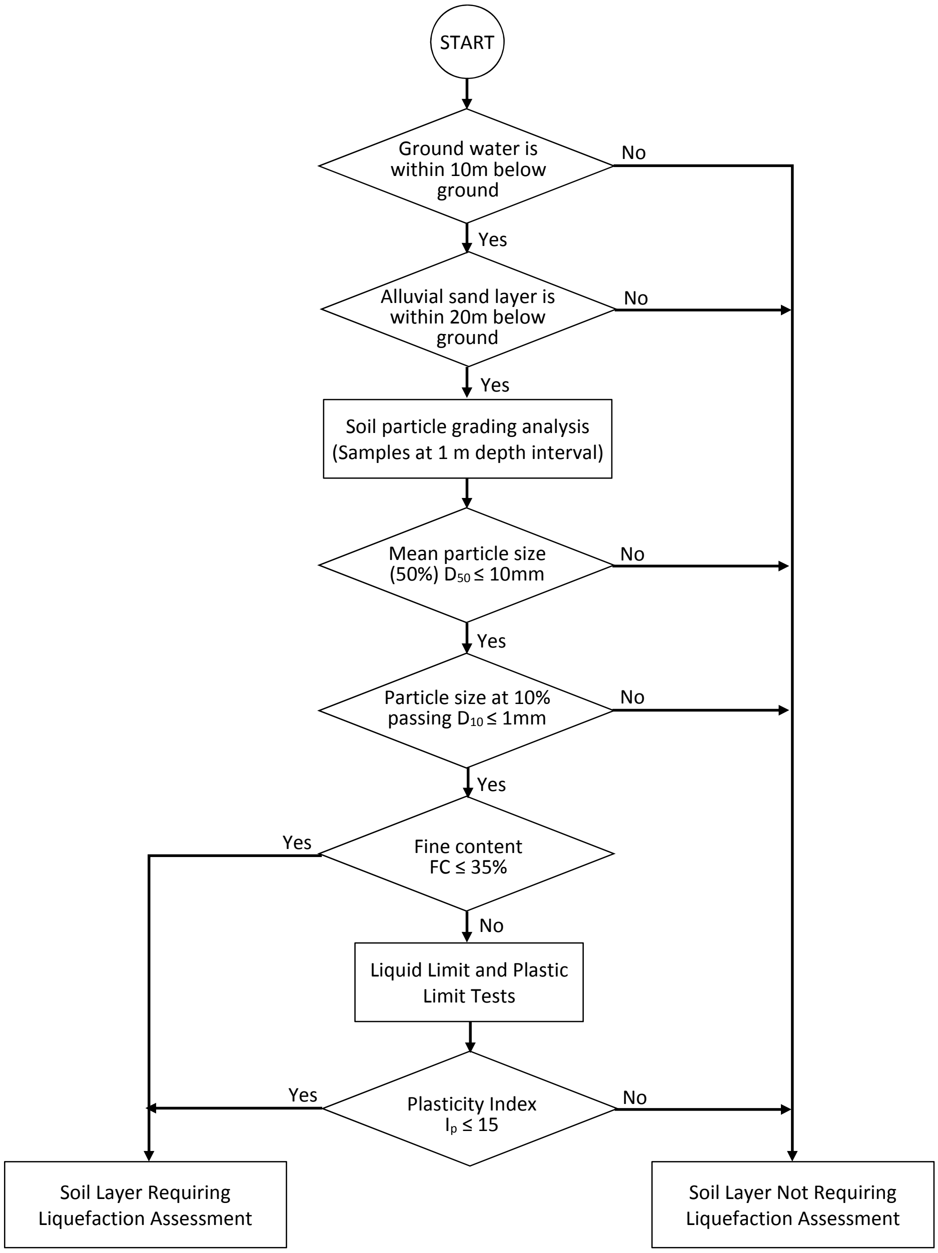


Figure C6.2.3-1 Determination of Necessity for Liquefaction Assessment of Soil Layer

2) The lower limit of the particle size of a soil layer requiring liquefaction assessment has been set in these Specifications according to the research results of recent years. In the past cases, most layers found to have liquefied had an *FC* of less than 35%. However, there were some cases in which liquefaction also occurred in the soil layers having an *FC* value over 35% and a low plasticity index, such as silt sandy soil. Therefore, the assessment standards have been provided in the above provisions. When the *FC* value is less than 35%, there is no need to perform the liquid and plasticity limit state testing.

3) With regards to the upper limit of the grain size of a soil layer requiring liquefaction assessment, since observations of the effects of recent earthquakes including the Hyogo-ken Nanbu Earthquake have revealed that liquefaction have occurred to gravelly soil with a mean diameter larger than 2 mm, the Specifications in this respect have been provided in the above provisions. However, the grain diameter indicated here is a value obtained by analyzing the grain size of the specimens for the standard penetration test. The specimens used for the standard penetration test have a finer grain size than the in-situ material due to the consequence of crushing during the test. Although this difference does not necessarily have anything to do with the hardness or coarseness of the grains, it can be assumed that the mean grain size diameter of 10 mm of a specimen used for the standard penetration test is roughly equivalent to the in-situ material with a mean grain size diameter of approximately 20 mm or more.

   The grain size diameter at 10% passing (D_{10}) was set to less than 1 mm due to the consideration that the gravelly soil having coarse grains and having a low uniformity coefficient is usually high in permeability of water and is therefore difficult to liquefy. Here, a distinction shall be made of the mean grain size diameters i.e. for sandy soil, D_{50} shall be less than 2 mm while for gravelly soil, D_{50} shall be more than 2mm.

   Up to now, there have been no cases of confirming liquefaction in diluvial soil during any earthquake, including the Hyogo-ken Nanbu Earthquake. Since the N value of diluvial soil is usually high and as a result of diagenesis, its resistance to liquefaction is also high, the possibility of liquefaction generally remains small in diluvial soil. However, there are some regions in which diluvial soil layer is found with a low N value or has lost its diagenesis ability. It is recommended that liquefaction assessment is carried out for such diluvial soil.

(2) Liquefaction assessment shall be performed for Level 2 Earthquake Ground Motion. Since the cyclic triaxial shear stress ratio R_L changes greatly according to the repeated property of seismic ground motions, the ratio shall be modified by Equation 6.2.3-8.

In case that the water level is above the ground surface as in a riverbed, the total overburden pressure and the effective overburden pressure shall be obtained assuming that the underground water level is on the ground surface. This is because water, which does not transmit shear force, does not act as an external force against the ground during an earthquake and because the load of the water above the ground surface does not contribute to an increase in the effective overburden pressure.

When considered particularly necessary, various testing and analyses can be performed such as the most detailed and up-to-date ground survey and testing at the site, soil testing in laboratory and response analysis of the ground. In addition to these, past data shall also be taken as reference for assessment of liquefaction.

(3) The equations used to calculate the cyclic triaxial shear stress ratio R_L as stipulated in these Specifications have been devised for sandy soil and gravelly soil separately based on the results of the laboratory undrained cyclic triaxial testing using frozen undisturbed specimens and the results of case studies including the Hyogo-ken Nanbu Earthquake.

As for the effect of particle size during evaluation of cyclic triaxial shear stress ratio R_L, modification of N-value based on the fine-grained fraction *FC* shall be considered for sandy soil. The reasons for this are as follows:

1) Concerning the effect of the property of grain size on the cyclic triaxial strength ratio, it has been concluded that sandy soil that is relatively fine-grained is greatly influenced by the fine-grained fraction *FC* and therefore the effect of grain size can be evaluated based on its fine-grained fraction *FC*.

2) There are two applicable methods when considering the effect of grain size on the cyclic triaxial shear stress ratio. One is to treat the effect of grain size as the increment of the cyclic triaxial shear stress ratio and the other is to regard it as the increment of the N-value. The second method can provide a more appropriate evaluation of the strength of the soil that contains a relatively high fine-grained *FC* and has a large N-value.

   It is recommended that a penetration test to measure the N-value is conducted by the free drop method which will result in a small loss of energy at the moment of impact. Besides, in Equation 6.2.3-14, since the modified N-value of gravelly soil is measured a little high due to the influence of the existence of gravel, the N-value obtained shall be reduced in accordance with the mean grain diameter D_{50} to estimate the cyclic triaxial shear stress ratio. However, since little data of this kind has been collected for gravelly soil, the N-value provided in Equation 6.2.3-14 can be used for the evaluation of the cyclic triaxial shear stress ratio for the time being.

   It has been argued that the cyclic triaxial shear stress ratio of the soil in a reclaimed land is lower than the value obtained by Equation 6.2.3-9. So far the data collection in this respect is not yet sufficient and the differences have not been clarified between the strength properties of this kind of soil and those of the alluvial soil. Therefore, no particular specifications have been provided concerning the soil in a reclaimed land. More survey and research work shall be necessary in this area.

6.2.4 Reduction of Geotechnical Parameters

(1) When a soil layer is considered to be an extremely soft layer according to the provisions specified in Article 6.2.2, its geotechnical parameters (shear modulus and strength) shall be assumed to be zero in the seismic design.

(2) For a sandy layer causing liquefaction and affecting a bridge according to the provisions of Article 6.2.3, the geotechnical parameters for the seismic design shall be reduced in accordance with the liquefaction resistance factor F_L.

Geotechnical parameters of a sandy layer causing liquefaction and affecting the bridge shall be equal to the product of geotechnical parameters obtained without liquefaction and the coefficient D_E in Table 6.2.4-1. For the case when $D_E = 0$, the geotechnical parameters (shear modulus and strength) shall be taken as 0 in seismic design.

(3) The weight of a soil layer with reduced or zero geotechnical parameter in the seismic design shall be assumed to be acting as an overburden.

Table 6.2.4-1 Reduction Factor D_E for Geotechnical Parameters

Range of F_L	Depth from Present Ground Surface x (m)	Dynamic Shear Strength Ratio, R	
		$R \le 0.3$	$0.3 < R$
		Verification for Level 2 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion
$F_L \le 1/3$	$0 \le x \le 10$	0	1/6
	$10 < x \le 20$	1/3	1/3
$1/3 < F_L \le 2/3$	$0 \le x \le 10$	1/3	2/3
	$10 < x \le 20$	2/3	2/3
$2/3 < F_L \le 1$	$0 \le x \le 10$	2/3	1
	$10 < x \le 20$	1	1

Commentary C6.2.4

(1) Since an extremely soft soil layer in seismic design cannot be counted on to provide strength and bearing capacity, its soil parameter shall be taken as zero in design.

(2) The strength and bearing capacity of soil decline in a liquefied sandy soil layer. Consequently, when liquefaction has been evaluated for a sandy soil layer according to the criteria of Article 6.2.3, the geotechnical parameter of that soil layer shall be multiplied by the coefficient D_E provided in Table 6.2.4-1 corresponding to the value of resistance factor F_L against liquefaction which is calculated by Equation 6.2.3-1. The geotechnical parameter to be used for seismic design shall be either zero or a reduced value.

Here, the geotechnical parameter reduced by multiplying D_E indicates the subgrade reaction coefficient, the upper limit value of the ground reaction and the maximum skin friction force. Even when F_L is below 1.0, the significance can be different when its value is 0.9 or 0.5. Therefore, by summarizing the analysis results of the damage cases caused by the Hyogo-ken Nanbu Earthquake and the effect on design etc., the values of the coefficient D_E to be multiplied by the geotechnical parameter have been designated as those shown in Table 6.2.4-1, which shall be correspondent to the values of F_L and the dynamic shear strength ratios R. The reason why the values of D_E shall vary according to the values of R is that even if the values of F_L are the same, if the value of R is high, the decrease in subgrade reaction will be small compared with the case in which the value of R is low.

F_L is obtained at the depth for which the standard penetration test is performed, but in order to obtain D_E, F_L shall usually be calculated at an interval of about 1 m to find the mean value of F_L for each soil layer. Then D_E can be obtained according to Table 6.2.4-1. Additionally, considering that the deeper the ground, the smaller its vibration will become and that there are very few cases of complete liquefaction in the soil layers deeper than 10 m, the values of the coefficient D_E shall be changed within the boundary of 10 m below the ground surface.

(3) For a soil layer whose geotechnical parameter is set as zero or has been reduced in seismic design, if there is no possibility that the soil will ever be excavated or scoured in the future, the soil layer shall be assumed as a load weight acting upon the ground below it. Consequently, for a soil layer whose geotechnical parameter is set as zero or has been reduced in seismic design, when the bearing capacity of the bottom surface of a footing is calculated, this weight shall have to be considered.

Moreover, for a soil layer whose geotechnical parameter is set as zero or has been reduced in seismic design, it is acceptable that the seismic hydrodynamic pressure and seismic earth pressure are not considered as effects of earthquake. However, when it has been estimated that liquefaction-induced lateral spreading affecting the bridge will possibly occur, Article 6.3 shall be referred to for verification.

6.3 VERIFICATION OF SEISMIC PERFORMANCE OF FOUNDATIONS FOR LIQUEFACTION-INDUCED LATERAL SPREADING

6.3.1 General

(1) Ground with possible lateral movement

A ground with both of the following two conditions shall be treated as a ground with possible lateral movement affecting the bridge:

1) Ground within a distance of less than 100 m from a water front in a shore area formed by a revetment with an elevation difference of 5 m or more between the water bottom and the ground surface behind.

2) Ground with a sandy layer thicker than 5 m that is assessed as a liquefiable layer according to the provisions in Article 6.2.3 and is distributed somewhat widely in the area of the water front.

(2) Verification of seismic performance of a bridge for liquefaction-induced lateral spreading

A pier foundation situated on a ground specified in Item (1) above shall be verified against possible liquefaction-induced lateral spreading. In the verification, lateral movement force specified in Article 6.3.2 shall act on the pier foundation. However, the lateral movement force and the inertia force need not to be considered simultaneously.

Commentary C6.3.1

(1) Since liquefaction-induced ground flow occurs in response to a decline in the bearing capacity accompanied by liquefaction, it can be assumed that liquefaction-induced lateral spreading will possibly occur in a soil layer for which liquefaction has been estimated according to the requirements of Article 6.2.3 and which at the same time is subjected to the action of asymmetrical earth pressure. During the Hyogo-ken Nanbu Earthquake of 1995, it was found that in some cases liquefaction-induced lateral spreading accompanied by liquefaction happened near the water front line in the shore area and that residual displacement of pier foundations occurred. The analysis results of the pier foundations that suffered residual displacement have revealed that the soil layers near the ground surface that did not liquefy moved along with the liquefied soil layers below, applying considerable forces to the footing. It was a phenomena confirmed for the first time. Although the necessary conditions for the occurrence of liquefaction-induced lateral spreading that will affect the bridge are still not fully understood, the types of the ground for which liquefaction-induced lateral spreading affecting the bridge will possibly occur have been specified in the provisions based on the damage cases of the Hyogo-ken Nanbu Earthquake.

Here, an elevation difference of 5 m or more between the ground behind a revetment and the sea bottom in front of it has been included in the judgment criteria. This is because during the Hyogo-ken Nanbu Earthquake, the elevation difference between the ground behind the revetment and the sea bottom in front along the shore area where liquefaction-induced lateral spreading caused residual displacement in the pier foundations was more than 10 m. However, in some locations where liquefaction-induced lateral spreading occurred, smaller elevation differences were found. Also, the area range within which a possible liquefaction-induced lateral spreading affecting a bridge is likely to occur shall be within 100 m from the water front line. This has been set by referring to the range of the residual displacement that occurred to the pier foundations which were caused by the liquefaction-induced lateral spreading during the Hyogo-ken Nanbu Earthquake. The distance from the water front line indicates the distance from the water front line that is closest to the pier foundation concerned in the verification of seismic performance and it shall be determined according to Figure C-6.3.1-1.

A thickness of 5 m or more has been provided as one of the judgment criteria for the sandy layer for which liquefaction is estimated. This was decided with reference to the ground conditions at the locations where liquefaction-induced lateral spreading caused residual displacement in the pier foundations and at the locations where big ground displacement occurred during the Hyogo-ken Nanbu Earthquake. In addition, since liquefaction-induced lateral spreading is a phenomenon accompanied by the liquefaction of the ground in a large scale, liquefaction assessment for the areas between the water front line and the location of each foundation was conducted. Based on the results, it can be considered that even within 100 m from the water front line, as long as the layer for which possible liquefaction is estimated not to exist continuously in the horizontal direction from the water front line, liquefaction-induced lateral spreading affecting a bridge will not occur in the ground behind the water frontline.

Even in areas away from coastal regions, cases of bridge damage due to liquefaction accompanied by liquefaction-induced lateral spreading were found along the Shinano River in the City of Niigata during the Niigata Earthquake of 1964. This observation resulted to the beginning of consideration of liquefaction effect in seismic design. After that, the first case of a bridge being seriously affected by liquefaction-induced lateral spreading was identified in shore areas during the Hyogo-ken Nanbu Earthquake. The mechanism of liquefaction-induced lateral spreading and its effects on structures around river areas are usually considered to be different from the same phenomena happening in shore areas. However, if both of the conditions described in Items 1) or 2) in the above provisions are found to exist in the ground in a high water channel behind a vertical low water revetment, in which it is presumed to be strongly affected by the eccentric earth pressure or in the ground inside a vertical special dike, it is suggested that the effects of liquefaction-induced lateral spreading should be studied by following the standards for shore areas. To deal with liquefaction-induced lateral spreading around river areas, some measures shall be necessary based on the results of further investigations and researches in the future.

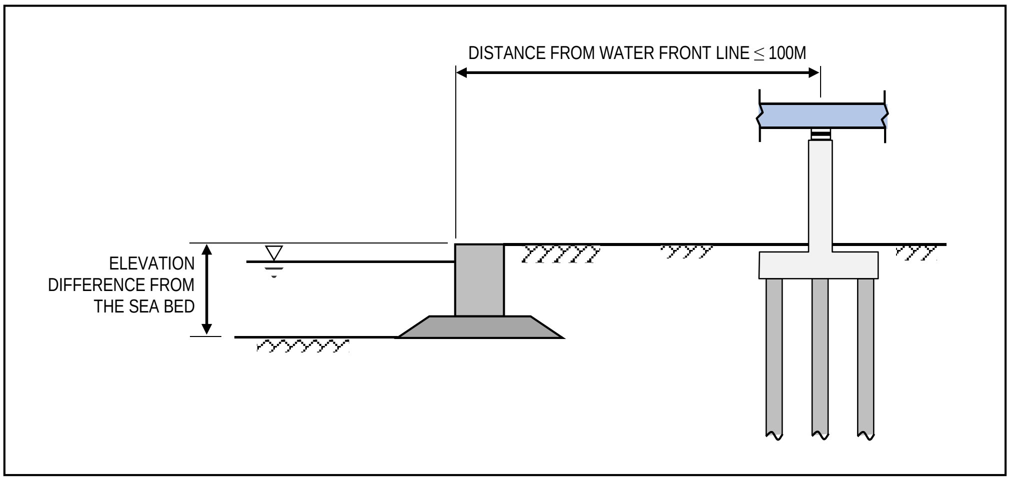


Figure C6.3.1-1 Elevation Difference from the Sea Bed and Distance from the Water Front Line

(2) It is a rule that during verification of seismic performance of a bridge with regard to liquefaction-induced lateral spreading, the lateral movement force specified in Article 6.3.2 shall be acting upon the pier foundation.

In the Hyogo-ken Nanbu Earthquake, damages due to liquefaction-induced lateral spreading were found in the abutment parapet, abutment body and pile heads. It would be an understatement to say that the mechanism of the effect of liquefaction-induced lateral spreading on a structure like an abutment subjected to asymmetrical earth pressure still remains unclear.

However, it is understood that since an abutment is a structure under asymmetrical earth pressure all the time, the movement of the abutment due to the influence of liquefaction-induced lateral spreading will occur in the push-forward direction, the possibility of this effect to cause the unseating of the superstructure is rather small. For this reason, it is acceptable if verification of seismic performance for an abutment considering the effect of liquefaction-induced ground flow is not carried out. When the abutment is likely to move forward, large displacements may occur to the piers in the middle of the bridge due to the superstructure being pushed out by the abutment. The influence of such a phenomenon has to be examined.

In addition, in a skew bridge and a curved bridge, if the abutment moves forward due to the influence of liquefaction-induced lateral spreading, the entire superstructure will possibly rotate to cause a large displacement. In this case, it is required that the effect of liquefaction-induced lateral spreading shall be a subject for study.

Although yielding of foundation is not basically allowed in these Specifications, the effects of liquefaction-induced lateral spreading may be very difficult to be resisted elastically by the foundation during Level 2 EGM. In such case, when the resulting foundation becomes impractical under the elastic range, the DPWH may allow yielding of the foundation for special bridge cases. DPWH approval is necessary when considering such foundation yielding. It has been stipulated that during verification of a pier foundation concerning the liquefaction-induced lateral spreading, the horizontal displacement at the top of the foundation shall not exceed the allowable displacement. Here, the horizontal displacement generally refers to the relative displacement from the center of rotation in the case of a caisson foundation, a steel pipe sheet pile foundation, and a cast-in-place diaphragm wall foundation. In case of a pile foundation, it shall be taken as the relative horizontal displacement from the pile tip. The allowable displacement has been determined as 2 times the horizontal displacement when the yield of the foundation specified in Article 5.4.1(2) has been reached. This is due to the reason that if the displacement of the foundation reaches 2 times the yield displacement, it can increase considerably merely with a small amount of load being added. On the other hand, there are always some uncertainties in the evaluation of the loads acting upon the pier foundations due to the influence of liquefaction-induced lateral spreading. In order to prevent a big displacement from occurring because of such uncertainties, the allowable displacement has been set as 2 times the yield displacement.

Although many aspects concerning the mechanism of occurrence of liquefaction-induced lateral spreading are still not well understood, it is reported that after an earthquake, liquefaction-induced lateral spreading begins at the stage when the excessive pore water pressure rises and liquefaction has advanced to a certain degree. Since it can be considered that in most cases, the major vibrations of the earthquake ground motions have already ended by this stage, it has been stipulated that the inertia force and the effects of liquefaction-induced lateral spreading do not have to be taken into account at the same time.

In case that liquefaction-induced lateral spreading affecting the bridge is likely to occur, it is important to consider measures which include not only reinforcement of the foundation, but also adoption of a type of foundation having a strong lateral stiffness so as to ensure the earthquake resistance capacity of the entire bridge.

6.3.2 Calculation of Lateral Force on Foundations by Liquefaction-Induced Lateral Spreading Force

When considering the effects of liquefaction-induced lateral spreading, lateral movement forces acting on pier foundations shall be obtained as follows:

In the case where lateral movement occurs under the conditions shown in Figure 6.3.2-1, lateral movement forces per unit area calculated by Equations 6.3.2-1 and 6.3.2-2 shall be applied to structural members in the non-liquefying layer and those in the liquefying layer, respectively, both of which are located above the depth in consideration of the effects of lateral movement. In this case, the horizontal resistance of the soil layer within the thickness necessary to consider the effects of lateral movement cannot be expected.

$$q_{NL} = c_s\, c_{NL}\, K_p\, \gamma_{NL}\, x \qquad (0 \le x \le H_{NL}) \qquad \text{(6.3.2-1)}$$

$$q_L = c_s\, c_L \{\gamma_{NL} H_{NL} + \gamma_L (x - H_{NL})\} \qquad (H_{NL} < x \le H_{NL} + H_L) \qquad \text{(6.3.2-2)}$$

where

q_{NL} : Lateral movement force per unit area (kN/m^2) acting on a structural member in a non-liquefying layer at the depth x (m).

q_L : Lateral movement force per unit area (kN/m^2) acting on a structural member in a liquefying layer at the depth x (m).

c_s : Modification factor on distance from the water front. The value of c_s shall be taken from Table 6.3.2-1.

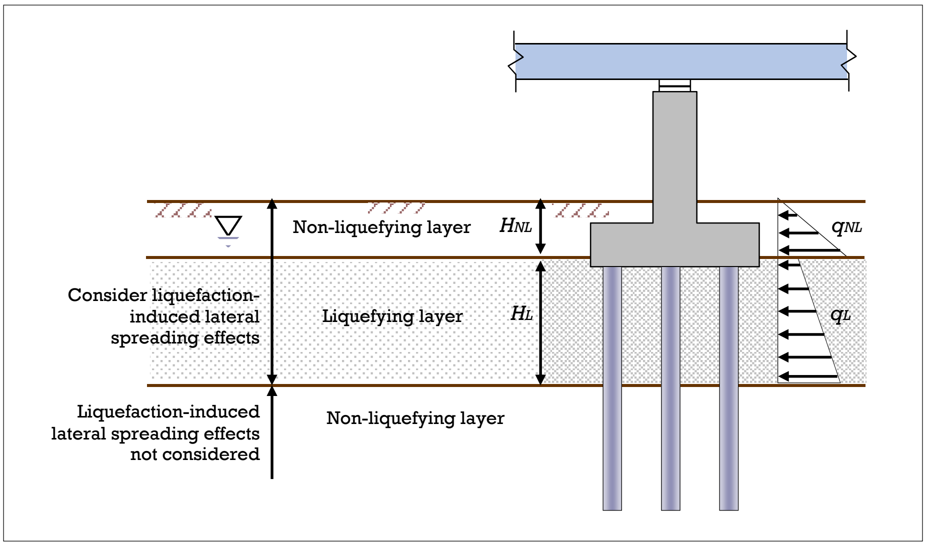


Figure 6.3.2-1 Model for Calculating Lateral Movement Forces

c_{NL} : modification factor of the lateral movement force in a non-liquefying layer. The value of c_{NL} shall be taken from Table 6.3.2-2, according to liquefaction index P_L (m^2) obtained from Equation 6.3.2-3.

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5x)\,dx \qquad \text{(6.3.2-3)}$$

c_L : Modification factor of the lateral movement force in a liquefying layer (can be taken as 0.3).

K_p : Passive earth pressure coefficient (under normal condition).

γ_{NL} : Mean unit weight of a non-liquefying layer, (kN/m^3).

γ_L : Mean unit weight of a liquefying layer, (kN/m^3).

x : Depth from the ground surface, (m).

H_{NL} : Thickness of non-liquefying layer, (m).

H_L : Thickness of liquefying layer, (m).

F_L : Liquefaction resistance factor calculated by Equation 6.2.3-1. If $F_L \geq 1$, $F_L = 1$.

Table 6.3.2-1 Modification Factor c_s on Distance from Water Front

Distance from Water Front, s (m)	Modification Factor c_s
$s \leq 50$	1.0
$50 < s \leq 100$	0.5
$100 < s$	0

Table 6.3.2-2 Modification Factor c_{NL} of Lateral Movement Force in Non-Liquefying Layer

Liquefaction Index P_L (m^2)	Modification Factor c_{NL}
$P_L \leq 5$	0
$5 < P_L \leq 20$	$(0.2\,P_L - 1)/3$
$20 < P_L$	1

Commentary C5.3.2

Although some aspects of the mechanism of the effects of liquefaction-induced lateral spreading on the pier foundation still need further research, a modeling method has been described in the provisions to deal with the liquefaction-induced lateral spreading forces acting upon the pier foundation due to the influence of liquefaction-induced lateral spreading. This model was established based on the results of damage analysis of the bridges on the reclaimed land in the shore areas during the Hyogo-ken Nanbu Earthquake. As illustrated in Figure 6.3.2-1, there is a soil layer that does not liquefy (the non-liquefying layer) close to the ground surface and a layer that does liquefy below it (the liquefying layer). The model has been created to show the necessary ranges for consideration of the influence of liquefaction-induced lateral spreading for both liquefying layer and non-liquefying layer. Consequently, when the conditions in design differ greatly from the model shown in the figure, appropriate modifications shall have to be made to the model. Besides, in case that the liquefying layer and the non-liquefying layer exist in an alternating pattern, Figure C6.3.2-2 shall be referred to for the necessary ranges of consideration of the influence of liquefaction-induced lateral spreading.

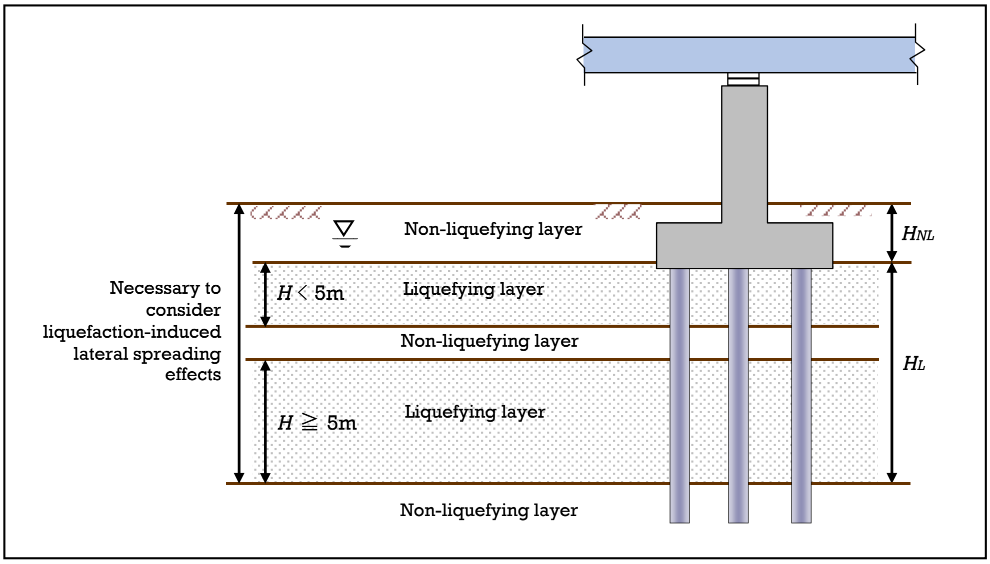


Figure C6.3.2-2 Example of the Necessary Ranges for Consideration of Liquefaction-Induced Lateral Spreading Effect When a Liquefying Layer and a Non-Liquefying Layer Exist in an Alternating Pattern

Based on the force corresponding to the passive earth pressure of a non-liquefying layer, Equation 6.3.2-1 is used to obtain the liquefaction-induced lateral spreading force per unit area acting on the structure from the non-liquefying layer. Concerning the relationship between the liquefaction-induced lateral spreading force and the distance from the water front line, the relationship between residual displacement in the piers and the distance from the water front line observed after the Hyogo-ken Nanbu Earthquake was taken as a reference to provide the modification factors in relation to the distances as shown in Table 6.3.2-1. When the quantity of liquefaction-induced lateral spreading is small, the liquefaction-induced lateral spreading force does not reach the passive earth pressure, but when the quantity increases to a certain extent, the liquefaction-induced lateral spreading force will reach the passive earth pressure for the first time. The coefficient c_{NL} has been given in Table 6.3.2-2 as a modification factor considering this phenomenon based on the inverse analysis of the pier foundations damaged by the liquefaction-induced lateral spreading during the Hyogo-ken Nanbu Earthquake.

On the other hand, Equation 6.3.2-2 provides the liquefaction-induced lateral spreading force per unit area acting on a structure from a liquefying layer based on the force equivalent to the total overburden pressure. The modification factor c_L of the liquefaction-induced lateral spreading force in a liquefying layer represents the percentage of the liquefaction-induced lateral spreading force acting upon the foundation out of the total overburden pressure. The value of c_L has been set as 0.3. The reason for this is that according to the inverse analysis results of the pier foundations suffering from the residual displacement which was caused by the liquefaction-induced lateral spreading during the Hyogo-ken Nanbu Earthquake, the liquefaction-induced lateral spreading force acting upon the foundations located inside the liquefying layer was about 30% of the total overburden pressure.

Since Equations 6.3.2-1 and 6.3.2-2 provide the liquefaction-induced lateral spreading force per unit area, the liquefaction-induced lateral spreading force per unit depth for a pier or footing can be obtained by multiplying the force by the width of the pier body or footing body respectively. Also, the liquefaction-induced lateral spreading force per unit area for a pile foundation shall be multiplied by the outermost widths between the piles at both ends of the surface resisting the liquefaction-induced lateral spreading force. The product thus obtained shall be treated as the liquefaction-induced lateral spreading force per unit depth of the pile foundation. For a foundation type other than a pile foundation, the value having been multiplied by its width can be regarded as the liquefaction-induced lateral spreading force per unit depth of the foundation concerned. For a pile foundation, sometimes a liquefaction-induced lateral spreading force passes the first row of piles and acts on the second and subsequent rows. The mechanism for this phenomenon is still not well understood. However, this effect is approximately incorporated here considering the projection area of the outermost edge. In addition, for design purpose, it can be assumed that all piles share the role of resisting the liquefaction-induced lateral spreading force.

In a case that there is no non-liquefying layer above the liquefying layer, when calculating the liquefaction-induced lateral spreading force for the ground that liquefies up to the ground surface, it will be sufficient to consider Equation 6.3.2-2 while ignoring Equation 6.3.2-1. In case of complicated ground conditions, the displacement caused by liquefaction-induced lateral spreading can be estimated by using the finite element analysis etc. when considering the effects of the liquefaction-induced lateral spreading.

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SECTION 7:
REQUIREMENTS FOR UNSEATING PREVENTION SYSTEM

7.1 GENERAL

(1) An unseating prevention system consists of the seating length of the girder at the support, unseating prevention device, device limiting excessive displacement, and device to prevent the superstructure from settling (limiting vertical gap in superstructure). These components shall be appropriately selected in accordance with the bridge type, type of bearing supports, ground conditions, and other factors deemed necessary by DPWH.

(2) For the unseating prevention systems in the bridge axis, the seating length specified in Article 7.2 and unseating prevention device specified in Article 7.3 shall be provided at the end supports and at the intermediate joints. Unseating prevention device can be omitted for a bridge in which longitudinal displacement is unlikely to occur in the bridge axis due to its structural characteristics, except those bridges specified in 1) and 2) of Article 7.5(1), or those located on seismically unstable ground, as specified in Section 6.

(3) However, for the unseating prevention systems in the transverse direction to the bridge axis, structures limiting excessive displacement shall be installed at end supports and at intermediate joints of the bridges specified in Article 7.5(1), and at intermediate supports of continuous girders of the bridges specified in Article 7.5(2).

(4) Special consideration, including the introduction of a structure or device to prevent the superstructure from settling (vertical gaps at superstructure due to damage of bearing supports) as specified in Article 7.4, shall be given when a tall bearing support is used for Critical (OC-I) and Essential (OC-II) bridges specified in Article 3.2.

Commentary C7.1

(1) Unexpected seismic force, displacement or deformation may occur in a bridge caused by unpredicted earthquake ground motion in the design, failure of the surrounding ground, or unexpectedly complicated vibrations in the structural members. This Section corresponds to establishing an unseating prevention system as a fail-safe structure against these unanticipated situations.

Various damages to the unseating prevention systems experienced in the Hyogo-Ken Nambu Earthquake (Japan) in 1995 are well known. Based on this experience, this Section clarifies what role each component comprising the unseating prevention system shall play, and specifies how to determine these components, including seating length, unseating prevention device, device limiting excessive displacement, and device to prevent the superstructure from settling. The essential functions of each component, as illustrated in Figure C7.1-1 are defined as follows:

1) Seating Length

The seating length should be long enough to prevent falling and unseating of the superstructure from the top of the substructure even in the event of an unexpectedly large relative displacement generated between the superstructure and the substructure.

2) Unseating Prevention Device

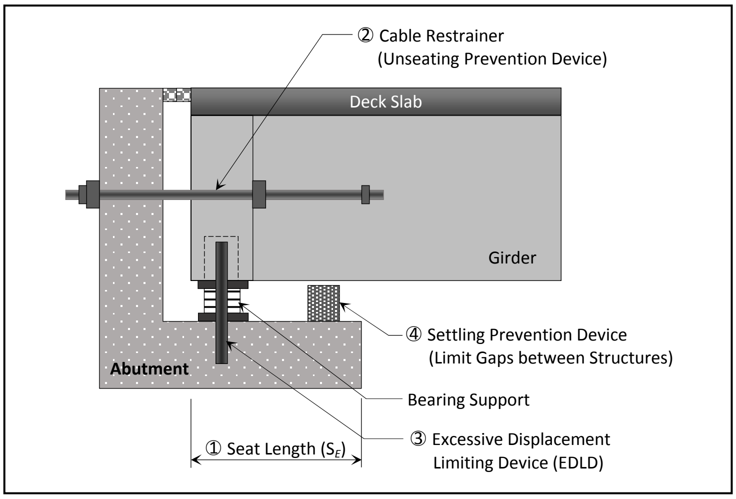
The unseating prevention device shall prevent the relative displacement in the superstructure from exceeding the seat length in the event of an unexpectedly large relative displacement generated between the superstructure and the substructure. At least 25% of the seat length should remain in the event of an unexpectedly large earthquake.

3) Device Limiting Excessive Displacement

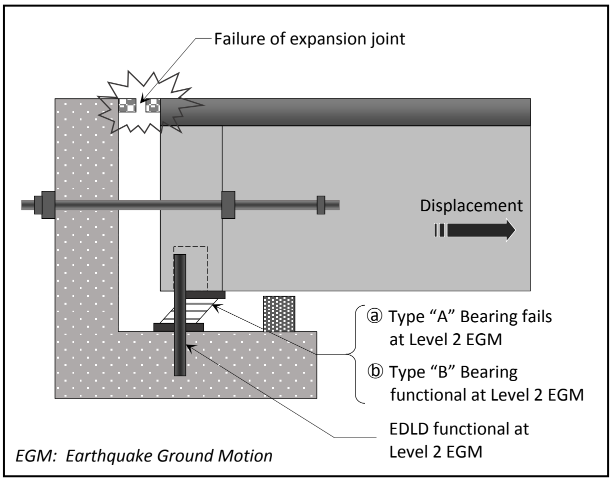
The device limiting excessive displacement is designed to resist inertial force for Level 2 Earthquake Ground Motion together with Type A (JRA) bearing supports. Relative displacement between the superstructure and the substructure shall be limited when bearing supports are damaged.

4) Structure to Prevent the Superstructure from Settling

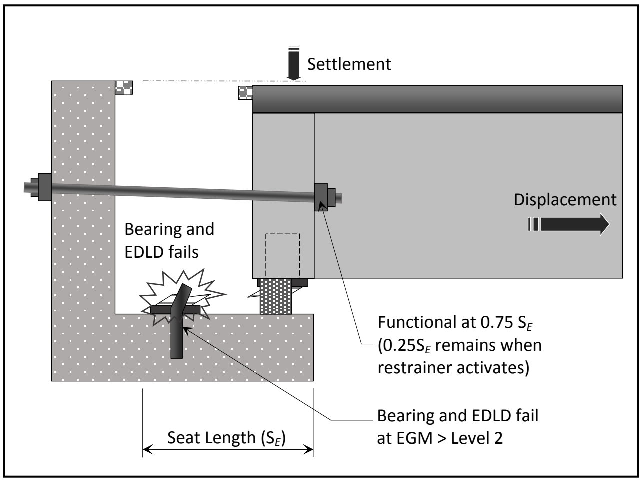
This is installed to prevent the occurrence of a gap that may hinder traffic over the bridge when the high bearing supports fail or are damaged.



(a) Layout of Unseating Prevention System



(b) Performance under design earthquake (Level 2 EGM)



(c) Performance under unexpected earthquake (EGM > Level 2)

Figure C7.1-1 Mechanism of Unseating Prevention System

Figure C7.1-2 shows the fundamental principles related to the selection of unseating prevention system, which is mainly applied to bridges in which the superstructure and the substructure are connected through bearing supports, such as typical girder bridges, deck-type arch bridges and skew bridges, etc. For these bridges, special attention should be given to prevent unseating of the superstructures, taking into account the dynamic behavior of each bridge.

From the experience on past earthquake disasters, unseating prevention system of the following bridges need to be carefully investigated:

i. Bridges whose substructure is located on ground areas that is likely to deform

Substructures in this case may be largely deformed due to liquefaction or lateral spreading of the ground, and sliding of soft clay layers, so that the seating length and strength of the unseating prevention system are recommended to be determined from the results of the dynamic analysis.

ii. Bridges with substantially different substructure types or ground conditions

The dynamic behavior of the bridges during an earthquake may become complicated when different types of substructure are used in one bridge, or when the ground conditions are substantially different in substructures of the same type. Accordingly, the seating length shall be appropriately determined with reference to the results of dynamic analysis.

When selecting the structural types for bridges, appropriate measures shall be taken during the basic design phase, as it may be better to choose a continuous structure rather than having joint connections.

iii. Bridges having considerably different types or scales of adjoining superstructures

Large relative displacements may be generated in bridges with different types or scales of adjoining superstructures, because every design vibration unit moves at a different phase. In this case, an unseating prevention system that connects the adjoining superstructures together should be avoided. As specified in Article 7.2, the seating length should be long enough to include the effects of possible collision between girders based on the results of the dynamic analysis. Such bridges may be at least twice as heavy as the adjacent ones in weight, or have a natural period ratio greater than 1.5 between the two design vibration units.

iv. Bridges with very high piers

Longer seating length may be required for bridges with very high piers owing to long natural periods. The seating length of these bridges shall be determined from the results of the dynamic analysis in accordance with the provisions of Article 7.2.

v. Skew bridges and curved bridges

For skew bridges, some cases of complex structural behavior and response are observed as a result of the rotation of the superstructure around the vertical axis during an earthquake. In particular, the ends of superstructure in bridges with smaller skew angles may be unseated and therefore fall off the top end of the substructure due to rotation of the superstructure. Also, bridges with smaller curvature radius may sustain severe damage due to the rotation of the superstructure or displacement toward the outside of the curve. With regard to bridges of this type, seating length shall be determined based on the provisions of Article 4.3, Article 4.6 and Article 7.2 with reference to the results of dynamic analysis. In addition, unseating prevention systems shall be installed both in the longitudinal and transverse direction to the bridge axis in accordance with Article 7.5.

vi. Bridges with substructures that narrows at the top

Bridges with substructures narrowing at the top, such as those without coping beams in the piers are generally not preferable in seismic design because unseating may happen in the transverse direction once the bearing supports collapse. Special investigation should be made in accordance with the provisions of Article 7.2 with reference to the results of the dynamic analysis when this kind of substructure is adopted. In addition, unseating prevention systems shall be installed both in longitudinal and transverse direction to the bridge axis in accordance with Article 7.5.

vii. Bridges with fewer bearing supports in one bearing line

Bridges supported by one bearing support in one bearing line should install device or structure to limit excessive displacement both in the longitudinal and transverse direction to the bridge axis in accordance with Article 7.5. In such condition, the superstructure may unseat in the transverse direction once the bearing support collapses. In case where the spacing of Type A (JRA) bearing supports in one line of the bearing support is narrower than the height of the girder, it is desirable to install the unseating prevention system in the transverse direction to the bridge axis even though two bearing supports are placed in one bearing line.

(2) Seating length and unseating prevention device shall be provided for the unseating prevention systems in the bridge axis.

It is not necessary to install an unseating prevention device for bridges which have no tendency to displace in the longitudinal direction to the bridge axis due to their structural characteristics. Generally, these bridges are limited to those series of superstructures with lengths of up to 25 meters that are supported at both ends of abutments with high-stiffness and those which do not fall under the category 1) or 2) of Article 7.5. Also, this rule will apply to bridges that have a series of superstructures with a length of up to 50 meters each, when both ends of the abutments are supported in Type I Ground. Here, "series of superstructures" means superstructures on a single effective span or on multiple effective spans of a continuous or connected structure, excluding a series of simple bridges. Conversely, skew bridges with smaller skew angles or curved bridges with larger intersection angles shall be equipped with unseating prevention systems, consisting of appropriately selected elements, since large displacements may happen due to the effects of skew angles or intersection angles.

(3) Bridges that require a structure or device to limit excessive displacement for the unseating prevention system transverse to the bridge axis are specified in this section. Bridges that require unseating prevention measures transverse to the bridge axis include skew bridges, curved bridges, bridges with substructures narrowing at the top, and bridges with fewer bearings on one bearing line. However, skew bridges and curved bridges consisting of continuous girders are exempted from the requirement of installing structures that limit excessive displacements at the interior support, since transverse displacement will not happen when rotation of the girders is limited at the end supports.

(4) When a tall bearing support is used, both superstructure and substructure may be damaged once the superstructure drops off to the top of the substructure after the bearing supports are damaged during an earthquake with a large magnitude. Also, this may result in the development of a gap as large as several hundred millimeters on the road surface which may prevent emergency vehicles and other traffic from passing just after the event. For this reason, a special consideration, including an introduction of a structure or device to prevent the superstructure from settling, shall be given when tall bearing supports are used in critical and essential bridges to span an important route specified in Article 3.2.

On the other hand, when the height of the rubber bearings is comparatively small and their surface dimensions are relatively large compared with the height, the structure or device to prevent the superstructure from settling is not necessary. This is because the occurrence of a large gap in the level of the road surface can be prevented even when the bearing supports are damaged.

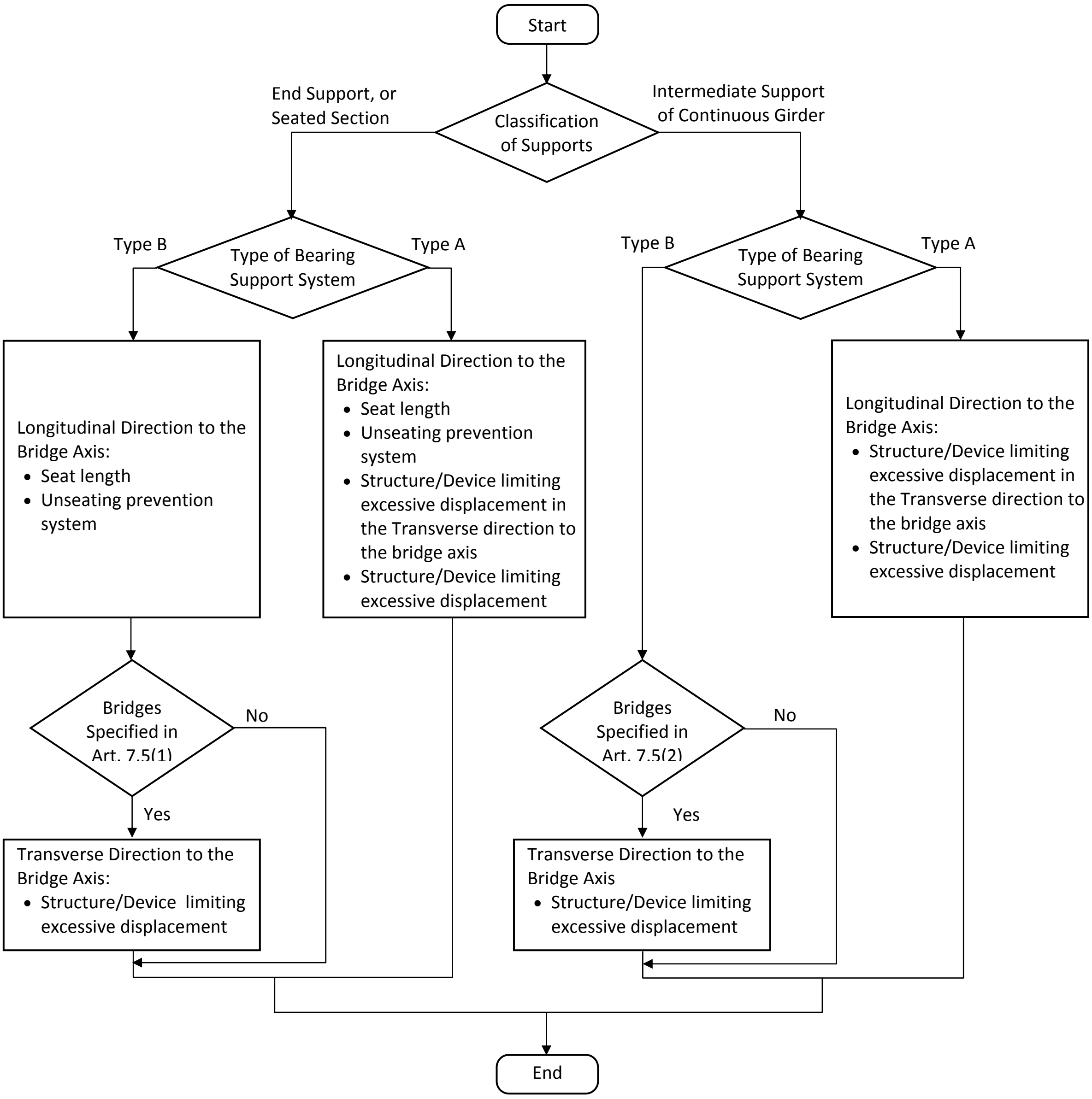


Figure C7.1-2 Fundamental Principles of Unseating Prevention System

7.2 SEAT LENGTH

(1) The seat length of a girder at its support shall not be less than the value obtained from either Equation 7.2-1 or Equation 7.2-2. Here, the seat length shall be measured in the direction perpendicular to the bearing support line when the direction of soil pressure acting on the substructure differs from the bridge axis, as in cases of a skew bridge or a curved bridge.

$$S_E = u_R + u_G \geq S_{EM} \quad \text{(7.2-1)}$$

$$S_{EM} = 0.70 + 0.005l \quad \text{(7.2-2)}$$

$u_G = \varepsilon_G L$ .... (7.2-3)

where:

S_E : Seat length of the girder at the support, (m). S_E is the length measured from the end of girder to the edge of the top of the substructure, or the girder length on the hinge/bearing joint, as shown in Figure 7.2-1.

u_R : Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 Earthquake Ground Motion, (m). In calculating u_R, the effects of the unseating prevention structure and the structure limiting excessive displacement shall not be considered. When soil liquefaction and lateral spreading as specified in Section 6 may affect displacement of the bridges, such effects shall be considered.

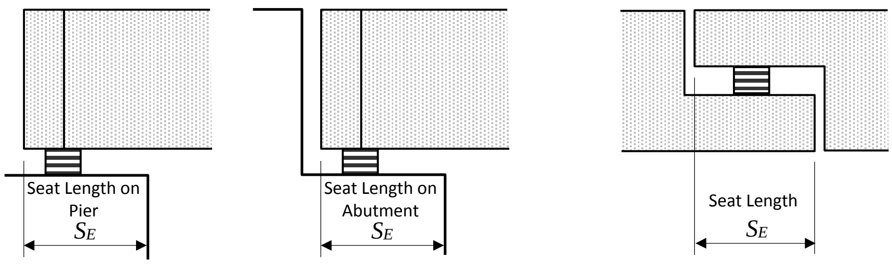
u_G : Relative displacement of the ground caused by seismic ground strain, (m).

S_{EM} : Minimum seating length of a girder at the support, (m).

ε_G : Seismic ground strain. ε_G can be assumed as 0.0025, 0.00375 and 0.005 for Ground Types I, II and III, respectively.

L : Distance between two substructures for determining the seat length (m).

l : Effective span length (m). When two superstructures with different span lengths are supported on one bridge pier, the longer of the two shall be used.



(a) Girder End Support (b) Intermediate Joint (Hinge/Bearing Joint)

Figure 7.2-1 Seat Length

(2) When verifying seismic performance of a bridge with complicated dynamic structural behavior, the maximum relative displacement obtained from the dynamic analysis shall be taken as u_R in Equation 7.2-1.

(3) For a skew bridge with superstructure shape satisfying Equation 7.5-1, the seat length shall satisfy the provisions in (1) and be calculated by Equation 7.2-4. For an asymmetric skew bridge in which the two front lines of the bearing supports at both ends of the superstructure are not parallel, $S_{E\theta}$ shall be calculated with the use of a smaller skew angle.

$S_{E\theta} \geq (L_\theta/2)(\sin\theta - \sin(\theta - \alpha_E))$ .... (7.2-4)

where:

$S_{E\theta}$: Seat length for a skew bridge, (m).

L_θ : Length of a continuous superstructure, (m).

θ : Skew angle, (degree).

α_E : Marginal unseating rotation angle, (degree). α_E can generally be taken as 5 degrees.

(4) For a curved bridge with superstructure shape satisfying Equation 7.5-2, the seat length shall satisfy the provisions in (1) and be calculated by Equation 7.2-5.

$$S_{E\phi} \geq \delta_E \frac{\sin\phi}{\cos(\phi/2)} + 0.3 \qquad \text{(7.2-5)}$$

$$\delta_E \geq 0.005\phi + 0.70 \qquad \text{(7.2-6)}$$

where:

$S_{E\phi}$: Seat length for a curved bridge, (m).

δ_E : Displacement of the superstructure toward the outside direction of the curve, (m).

ϕ : Fan-shaped angle by the two edges of a continuous girder of a curved bridge, (degrees).

Commentary C7.2

(1) The seat length shall be calculated according to the maximum relative displacement at the end of the girder and ground strain caused by Level 2 Earthquake Ground Motion. However, when the seat length calculated by this means is less than the value obtained from Equation 7.2-2, the seat length obtained from Equation 7.2-2 shall be used.

Since the relative displacement will occur at the end of the girder other than the deformation of the bridge as a whole involving bearing supports, columns, foundations, etc, the relative displacement u_R may be generally calculated instead as follows, depending on the conditions of bearing support at the support point where the seat length is to be calculated.

1) When the concerned support is held by elastic rubber bearings (Refer to Figure C7.2-1(a))

When the concerned support is elastically supported by Type B (JRA) rubber bearings specified in Article 5.7, such as seismic isolation bearings or horizontal force distributing bearings (elastic bearings), the lateral strength of a column shall be the product of the equivalent horizontal force times dynamic modification factor c_m defined in Article 8.3.3 when considering plastic behavior of the column. Similarly, u_R can be defined as the relative displacement between the superstructure and the substructure occurring at the support concerned when plastic behavior of the foundation is involved and the support is subjected to a horizontal force equal to product of a horizontal force equivalent to maximum response displacement times dynamic modification factor c_m.

u_R shall be taken as the elastic deformation of the rubber bearings, and be calculated by Equation C7.2-1 when the substructure containing the support concerned and the superstructure supported by it are considered to be in the same vibration unit.

$$u_R = \frac{c_m P_u}{k_B} \qquad \text{(C7.2-1)}$$

where:

u_R : Maximum relative displacement between the superstructure and the substructure due to Level 2 Earthquake Ground Motion, (m).

P_u : Horizontal force equivalent to the lateral strength of a column (or plastic shear) when considering plastic behavior of the column; or horizontal force equivalent to maximum response displacement of a foundation when considering plastic behavior of the foundation, (kN).

c_m : Dynamic modification factor specified in Article 8.3.3 and can be taken as 1.2.

k_B : Spring constant of the bearing support, (kN/m).

The seat length shall be calculated in the same way as supported by movable bearings specified in Article C7.2 3) when Type A rubber bearings specified in Article 5.7 are used, since the bearings may be damaged once an inertial force corresponding to Level 2 Earthquake Ground Motion occurs.

2) When the concerned support is held by fixed supports

When fixed supports of Type B bearing are used at the concerned support, one-half of the width of the bearing support shall be taken as the relative displacement u_R, while the whole width shall be used in case of fixed supports of Type A bearing. This is for preventing unseating of the bridge when the bearing support system is damaged unexpectedly. Although the fixed supports using Type B bearings are designed to be capable of transferring the superstructure inertia force caused either by horizontal force equivalent to the plastic shear or lateral strength of the column with plastic behavior and there is no relative displacement generated at the bearing support system under an assumed earthquake ground motion in the design, a minimum u_R shall be provided. In case the bearing support may be damaged for fixed supports utilizing Type A bearing, the u_R for Type A bearing is assumed to be greater than that of Type B bearing.

Moreover, the width of the bearing support shall be that equal to the width of the lower shoe in the bridge axis in the case of metal bearings, or the width of the bearing support in the same direction for rubber bearings.

3) When the support concerned is supported by movable bearings (Figure C7.2.1 (b))

The maximum relative displacement, occurring between the two design vibration units containing both the superstructure and substructure at the concerned support, should be equal to u_R which can be estimated by calculating the gap between the ends of superstructure described in Article 5.8 of these Specifications. However, the displacement shall be regarded as zero in case of the design vibration unit consisting of an abutment only, since high stiffness of the abutment towards back-filling often leads to a considerably small displacement in this direction due to the back-filling.

Generally, the relative displacement u_R can be calculated as follows:

$$u_R = \sqrt{\sum u_{Ri}^2} \quad (i=1,2) \qquad \text{(C7.2-2)}$$

$$u_{Ri} = u_{Pi} + u_{Fi} + u_{Bi} \qquad \text{(C7.2-3)}$$

$$u_{Pi} = u_{Ri}\, \delta_{yi} \qquad \text{(C7.2-4)}$$

$$u_{Fi} = \delta_{Fi} + \theta_{Fi}\, h_{0i} \qquad \text{(C7.2-5)}$$

$$u_{Bi} = c_m\, P_{ui} / k_{Bi} \qquad \text{(C7.2-6)}$$

where:

u_R : Relative displacement defined, in Equation 7.2-1, (m).

u_{Ri} : Displacement of the i-th design vibration unit shown in Figure C7.2-1(b), (m).

u_{Pi} : Response displacement of the column representing the i-th design vibration unit, (m).

u_{Fi} : Horizontal displacement at the height of superstructure inertial force due to displacement of the pier foundation representing the i-th design vibration unit, (m). The calculation shall include these effects when liquefaction and lateral spreading of the ground specified in Section 6 occurs.

u_{Bi} : Relative displacement at the bearing support system representing the i-th design vibration unit, (m).

μ_{Ri} : Response ductility factor of the column representing the i-th design vibration unit.

δ_{yi} : Yielding displacement of the column representing the i-th design vibration unit, (m).

δ_{Fi} : Response horizontal displacement of the pier foundation representing the i-th design vibration unit, (m).

θ_{Fi} : Response rotation angle of the pier foundation representing the i-th design vibration unit, (rad).

h_{0i} : Height from the ground surface of seismic design to the superstructure inertial force in the column representing the i-th design vibration unit, (m).

P_{ui} : Horizontal force equivalent to the plastic shear or lateral strength of a column with plastic behavior, representing the i-th design vibration unit, (kN).

c_m : Dynamic modification factor specified in Article 8.3.3 and can be taken as 1.2.

k_{Bi} : Spring constant of the bearing support for the i-th design vibration unit, (kN/m).

For bridges with very high piers, the seat length calculated from the above may be quite large but it should generally be ensured. In case the seat length is unreasonably excessive for a bridge structure, other structural consideration can be applied taking into account the results of dynamic analysis, including either increasing the substructure's stiffness, installation of unseating prevention device that would control excessive displacement, or structure or device limiting excessive displacement.

The relative displacement due to seismic ground strain shall be calculated by Equation 7.2-3. Equation 7.2-3 is intended to calculate seismic ground strain, assuming 0.25%, 0.375% and 0.5% for Ground Types I, II and III, respectively. This assumption is based on the past records of large earthquakes in which large deformations such as cracks on the surface of the ground occurred when the seismic ground strain was smaller than the values above, and therefore additional coverage is included for safety. The length L between the substructures that may affect the seat length shall be the distance between the substructure supporting the girder at the support where the seat length is to be calculated and one that may primarily affect the vibration of the girder containing the support. Figure C7.2-2 shows an example of this case. For a bridge with different ground conditions, calculations shall be carried out based on the soft ground conditions than real ones.

In case that soil liquefaction and lateral spreading may affect the displacement of the bridge, the horizontal displacement occurring at the height of the superstructure inertia force caused by displacement in the pier foundation with the flow pressure applied in accordance with the provisions of Article 6.3 shall be calculated using Equation C7.2-5. In such condition, the seat length shall be calculated for three cases: 1) lateral spreading occurs, 2) only liquefaction occurs, and 3) neither lateral spreading nor liquefaction occurs, and the largest one shall be used for the design. At least 50 cm should be added to the seat length as measures to cope with unexpected deformation caused by the flow pressure and the accuracy of calculation in case the horizontal displacement at the edge of the foundation exceeds the yielding displacement of the foundation. This is because the displacement may change sharply with a small change in loads once the foundation yields. However, when calculating the seat length with lateral spreading, the deformation of the bearing support, column or foundation caused by earthquake ground motion need not be considered at the same time.

Furthermore, the seat length shall be measured in the direction of the horizontal components of soil pressure, as shown in Figure C7.2-3 when the direction of the horizontal components of soil pressure is different from the bridge axis, as in a skew bridge. In a skew bridge, although the main axis of the girder is often not the same as that of the substructure, the vibrations in both longitudinal and transverse direction to the bridge axis will occur simultaneously. As a result, the seismic response behavior of the bridge becomes very complex. For this reason, the seat length in the direction of the minimum distance between the girder end and the edge of the top of the substructure, as illustrated in Figure C7.2-3, shall be taken as a safety measure.

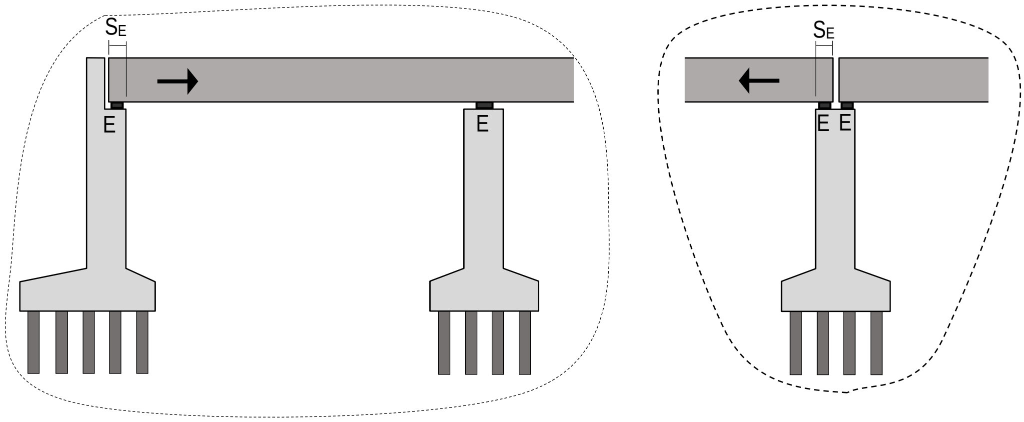
(2) When designing the seismic performance of the bridge by the dynamic method, the relative displacement at the girder end between the superstructure and substructure obtained from the dynamic analysis shall be applied to the design of the seat length.

(3) The seat length of a skew bridge shall be determined by considering the rotation of the superstructure, since unseating of the bridge may happen as a result of the structural behavior illustrated in Figure C7.2-4. For skew bridges with skew angle, length and width of a continuous superstructure satisfying Equation 7.5-1, including both simple girder bridges and continuous bridges, the bridge unseating may be caused by the rotation. In this case, the seat length $S_{E\theta}$ for rotation shall be determined based on limit of unseating rotation angle α_E, and the larger one obtained from Articles (1) or (2) of this commentary shall be taken as the seat length.

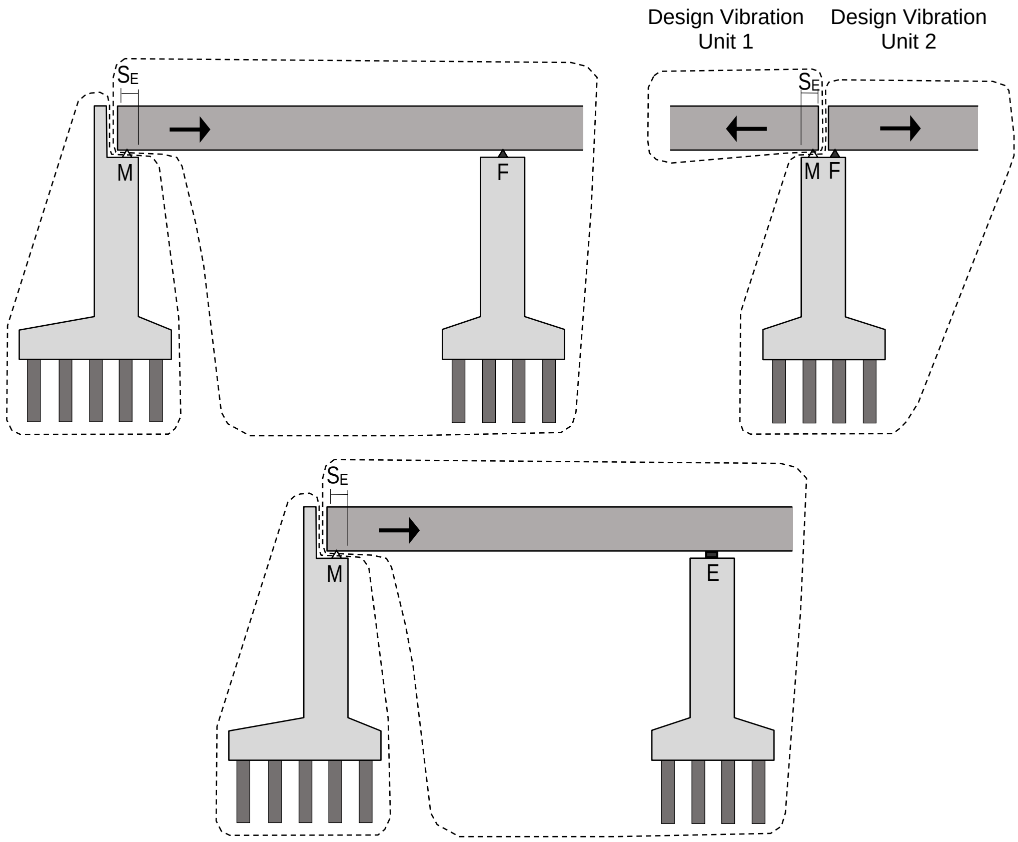
In Equation 7.2-4, only the skew bridge rotating around its center of gravity, which is limited by the unseating rotation angle, is included in which the center of the superstructure end comes out of the seat length. The limit of unseating rotation angle can generally be taken as 5 degrees, according to examples of damage during the 1995 Hyogo-Ken Nambu Earthquake (Japan), and the results of dynamic analysis for various bridge types using earthquake ground motions obtained from the event.

The limit of unseating rotation angle α_E shall be determined when the skew angle shall be made unavoidably narrower on a multi-span continuous bridge. For a multi-span continuous bridge with longer length of a continuous superstructure L_θ, the value of the seat length $S_{E\theta}$ obtained from Equation 7.2-4 may be quite large. In this case, the seat length may lead to a considerably irrational structure for the entire bridge. As a result, measures against such disadvantage should be taken, such as reexamining an appropriate limit of unseating rotation angle as described above, or making the structure limiting excessive displacement in the transverse direction to the bridge axis having the same strength as that of unseating prevention structure specified in Article 7.3 so that the rotational displacement of the superstructure could surely be restrained. In addition, the seat length in this case should generally be greater than 1.5 times the value obtained from Articles (1) or (2) of this commentary corresponding to different structural types and scales of the bridges.

The seat length shall be determined using the smaller skew angle, assuming the bridge rotating around the center line of the bridge axis, when the bearing lines on the two sides of the superstructure are not parallel, and when the skew angles at both ends are different.



(a) When the girder end is supported by rubber bearings



(b) When the girder end is supported by movable bearings

Figure C7.2-1 Inertia Forces Used in Calculating Seating Length

(4) For reason of structural characteristics shown in Figure C7.2-5, curved bridges are more prone to unseating due to rotation of the superstructure, or moving toward the outside of the curve. Consequently, the seat length shall be determined taking these effects into consideration. The superstructure may start rotating without being restricted by the abutment or the adjoining superstructures when the intersection angle, the length and the width of the series of superstructures satisfies Equation 7.5-2. In addition, the superstructure might move toward the outside of the curve line due to complicated vibrations occurring during an earthquake, accompanied by a danger of the

superstructure falling off the top of the substructure. Since the influence of the intersection angle is dominant during this type of movement, the limit of unseating rotation angle shall be determined from the intersection angle, and the seat length shall be calculated using Equation 7.2-5. Similarly, the seat length in the skew bridges obtained by this means shall be compared with the values obtained from Articles (1) or (2) of this commentary and the larger one shall be taken. When determining the seat length for continuous curved bridges, the intersection angle between the two girders may be large enough to consider careful attention to prevent superstructure rotation or movement toward the outside of the curve line of the superstructure.

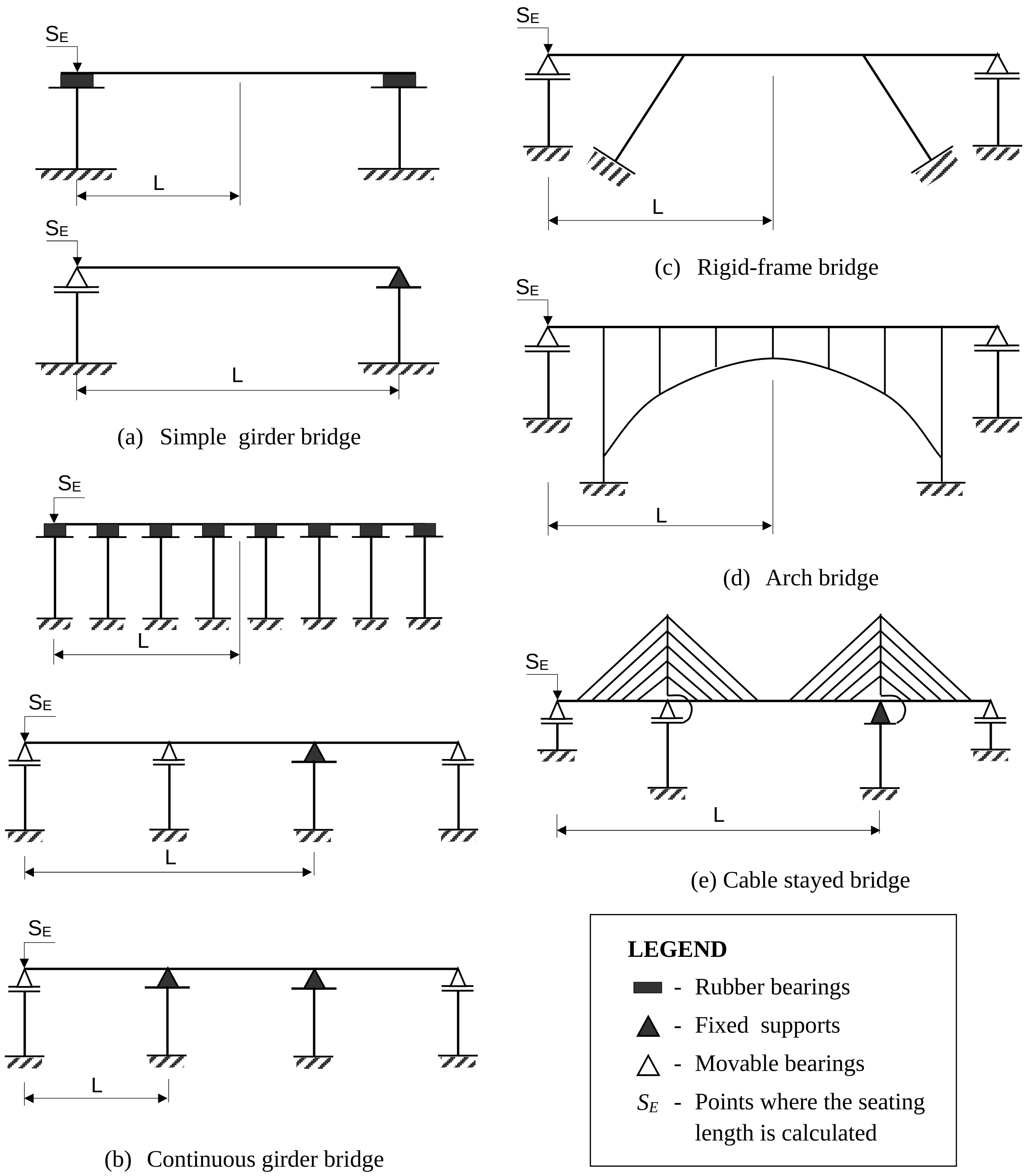


Figure C7.2-2 Measurement of Distance L between Substructures Affecting Seating Length

In case that the seating length $S_{E\phi}$, obtained from Equation 7.2-5 is large so that a considerably irrational structure in the entire bridge is formed, measures against such disadvantage should be taken so that the rotation or movement toward the outside of the curve line of the curved bridges can be properly restrained. Such measures include providing a device or structure limiting excessive displacement in the transverse direction to the bridge axis having the same strength as that of an unseating prevention device specified in Article 7.3. In addition, the seat length in this case should generally be greater than 1.5 times the value obtained from Articles (1) or (2) of this commentary corresponding to different structural types and scales of the bridges.

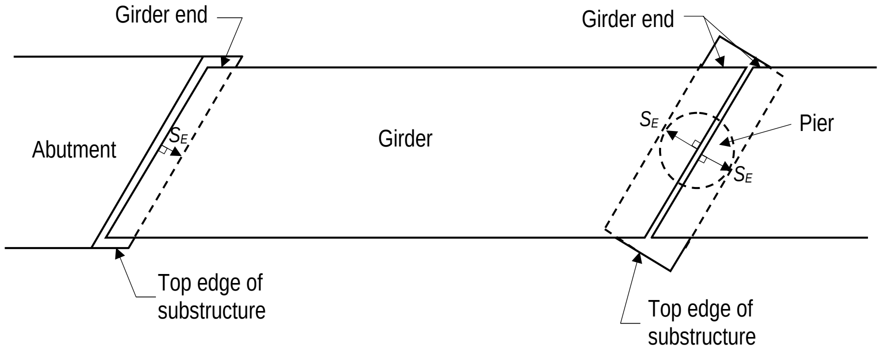


Figure C7.2-3 Measurement of Seating Length When Direction of the Horizontal Components of Soil Pressure is Different from the Bridge Axis

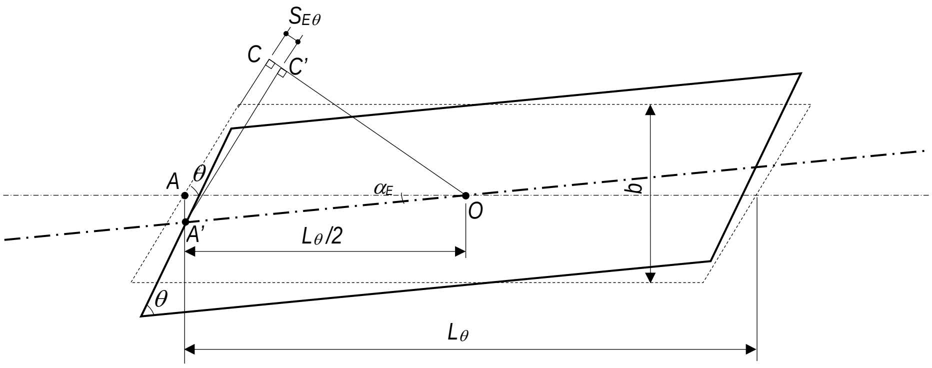


Figure C7.2-4 Seating Length of Skew Bridge

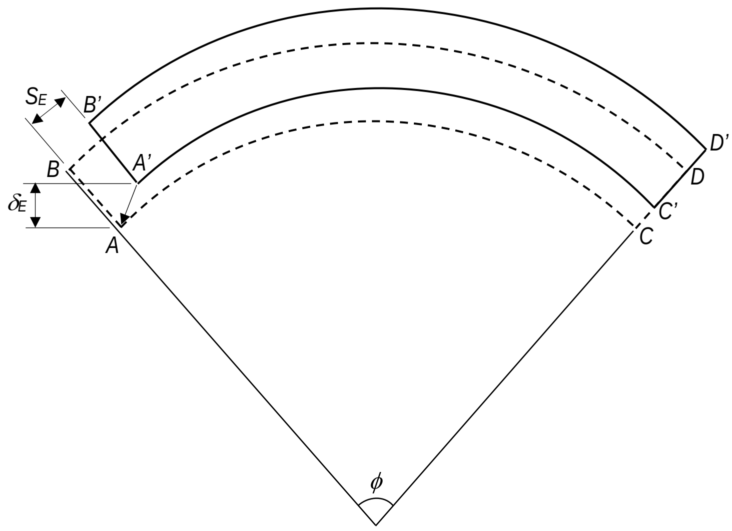


Figure C7.2-5 Seating Length Corresponding to the Movement of Curved Bridge

7.3 UNSEATING PREVENTION DEVICE (STRUCTURE)

(1) The ultimate strength of an unseating prevention device shall not be less than the design seismic force determined by Equation 7.3-1.

1) When the unseating prevention device directly connects the superstructure with the substructure, the design seismic force shall be:

$H_F = P_{LG}$

however, $H_F \leq 1.5\ R_d$ .... (7.3-1)

2) When the unseating prevention device connects the girders of the adjacent superstructures, the design seismic force shall be:

$H_F = 1.5\ R_d$

3) The design allowance length of the unseating prevention device should be taken as large as possible, but within the value given by Equation 7.3-2.

$S_F = c_F\ S_E$ .... (7.3-2)

where:

H_F : Design seismic force of the unseating prevention device, (kN).

R_d : Dead load reaction, (kN). In case the structure connects two adjacent girders, the larger reaction shall be taken.

P_{LG}: For abutments, this shall be the lesser value corresponding to the lateral (horizontal) capacity of the breast wall calculated from its nominal flexural resistance, or the nominal shear resistance of the breast wall, (kN), and

For columns/piers, this corresponds to the lateral (horizontal) strength of the column/pier based on the overstrength capacity or the shear forces resulting from plastic hinging of the column/pier, (kN).

S_F : Maximum design allowance length of the unseating prevention device, (m).

S_E : Seat length specified in Article 7.2, (m).

c_F : Design displacement coefficient of the unseating prevention structure. The standard value is taken as 0.75.

(2) The unseating prevention device shall not impair the functions of bearing supports, such as translational and rotational movements of the supports.

(3) The unseating prevention device shall be capable of moving in the transverse direction to the bridge axis and also alleviating the seismic impacts.

(4) A structure or joint connection of an unseating prevention device to the superstructure or the substructure shall be capable of transferring its design seismic force to both the superstructure and the substructure.

(5) The unseating prevention device shall allow maintenance and inspection of the bearing supports.

Commentary C7.3

(1) Unseating prevention structures are intended to supplement the seat length by functioning before the seat length is reached, when an unexpected relative displacement between the superstructure and the substructure occurs, as a result of the damage to the substructure or the bearings. Accordingly, the seat length greater than the displacement when the unseating prevention device just begins to function may not be necessary in this case, however, both the unseating prevention device and the seat length should be provided to ensure a fail-safe function against unexpected foundation failure or unusual structural damage.

In general, under extraordinary conditions such as an earthquake event that is greater than the design earthquake, the bearings and the devices that limits excessive displacements may fail. This behavior greatly alters the load intensity and the load transfer from the superstructure to the substructure. Under such situation, the unseating prevention device shall have the capacity to prevent the superstructure from falling down from the pier or abutment supports.

Basically, the design forces for the unseating device is taken as 1.5 times the dead load reaction ($1.5R_d$), when the unseating device directly connects adjacent superstructures (such as girder to girder connection) or when the unseating device connects the superstructure directly to the substructure. However, in some cases when the unseating device directly connects the superstructure with the substructure, the horizontal design force taken as $1.5R_d$ may result to unnecessarily oversized or over specified unseating device (especially for retrofit or maintenance design). In this case, the design forces for the unseating prevention device (given as P_{LG} in Equation 7.3-1) can be taken as:

- When the unseating device is directly connected to the abutments, the design force shall be the lesser value corresponding to the lateral (horizontal) capacity of the breast wall calculated from its nominal flexural resistance, or the nominal shear resistance of the breast wall, or
- When the unseating device is directly connected to the columns/piers, the design force shall correspond to the lateral (horizontal) strength of the column/pier based on the overstrength capacity or the shear forces resulting from plastic hinging of the column/pier.

However, either design forces calculated above shall not be greater than $1.5R_d$. Careful analysis shall be done to determine the member strengths or capacities considering the structural details and actual arrangement or reinforcing bars.

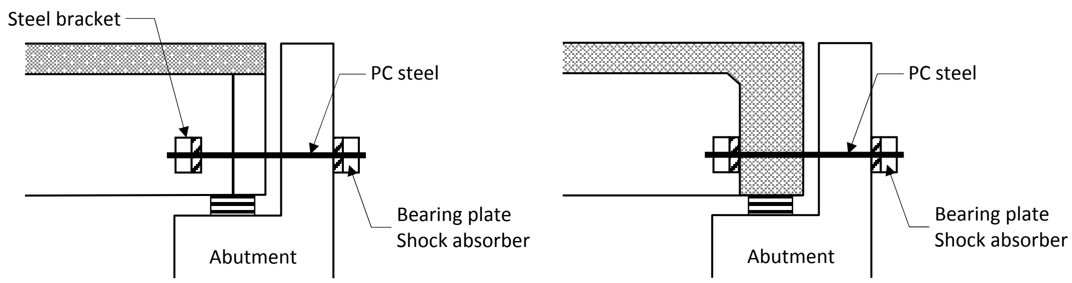
In addition, the design allowance length of the unseating prevention device S_F shall be taken as large as possible, but within the value given by Equation 7.3-2. This is because the overall structural damage to a bridge would be decreased when a certain amount of displacement in the bridge as a whole were allowed, and S_F smaller than the seating length S_E shall be taken simultaneously. For this reason, the standard value of design displacement coefficient c_F of unseating prevention device can be taken as 0.75. However, a smaller c_F can be applied when the S_E obtained from Equation 7.3-1 is large enough to provide a longer allowance length of the unseating prevention device.

The unseating prevention device is generally a structure of 1) connecting the superstructure and the substructure, 2) providing protuberances (e.g. shear block/shear key) either in the superstructure and the substructure, 3) joining two superstructures together. Examples of the unseating prevention devices are shown in Figure C7.3-1 to Figure C7.3-3. Actual examples include a device connecting the superstructure and the substructure with prestressing steel or anchor bars, or a device providing protuberances or unseating shear blocks/barrier walls on the top of the substructure and/or the superstructure, and a device connecting two series of superstructures with prestressing steel cable/rods.

Meanwhile, either the unseating prevention device or the device limiting excessive displacement shall generally not be used for both purposes since they have different functions even though they tend to have similar structures. Each function should be ensured in the above case. Loss of one function may lead to the loss of another if one structure also serves the function of the other one. On the other hand, these structures for both purposes may be used if the functions of both can be ensured independently.

(2) This provision described that unseating prevention devices shall not restrict other functions such as the movement or rotation of the bearing supports caused by an earthquake, change in temperature of the superstructure, or live load. However, when Type B rubber bearings are used, the movement corresponding to the allowable shear strain of the rubber should be ensured so as to use the full ductility of the rubber bearings during an earthquake with a large magnitude.

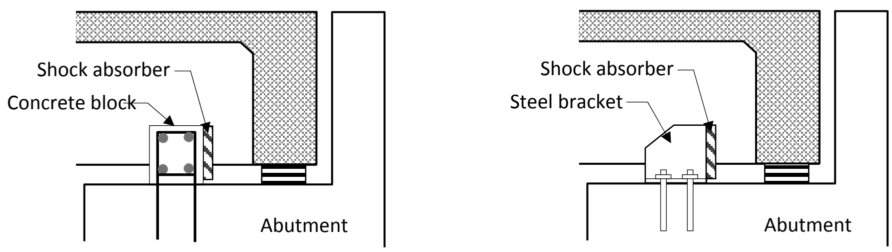
(3) According to examples of past disasters, many cases of damages have been observed accompanied by transverse displacement to the bridge axis with some seem to have resulted from the impact of a seismic force to the unseating prevention device. Therefore, the unseating prevention device shall be capable of following the movement transverse to the bridge axis with higher impact strength by employing a shock absorber such as rubber pad to alleviate the impact of the seismic force.



(a) Example of steel superstructure

(b) Example of concrete superstructure

Figure C7.3-1 Examples of Unseating Prevention Structures Connecting the Superstructure with the Substructure



(a) Use of concrete block

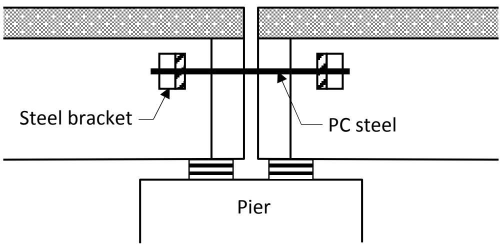
(b) Use of steel bracket

Figure C7.3-2 Examples of Unseating Prevention Structures Providing Protuberances on the Superstructure with the Substructure

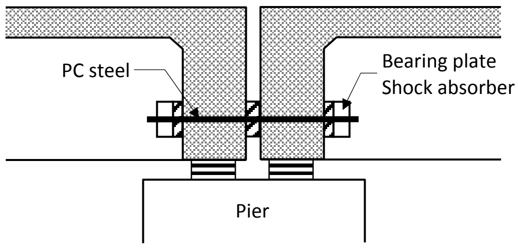
(4) Based on the damages caused by the 1995 Hyogo-Ken Nambu Earthquake, many damaged components were found not only in the unseating prevention devices but also in the connection areas of the superstructure and substructure. Unless the unseating prevention devices are confirmed for safety, including the section at which these are connected to the bridge, they cannot perform a fail-safe function to prevent the unseating of the bridge. For this reason, the connecting structure shall be

capable of securely transmitting the design seismic force applied to the unseating prevention device, and its safety shall also be verified in accordance with the requirements for steel bridges, concrete bridges and substructures with consideration of the design seismic force specified in Article (1) of this commentary. For instance, the parapet or backwall of an abutment should be properly designed when it is fixed to the unseating prevention device. Similarly, the requirements for concrete or steel structures can be applied to fixing the main girder, cross beam or partition to the unseating prevention device in connecting portions of T-girder bridges with concrete superstructure, box girder bridges or composite bridges. These components include the connections by prestressing steel, protuberance and its connection to the superstructure and the substructure under it. In addition, the portion at which the unseating prevention device is connected shall be capable of distributing the seismic force, over as wide an area as possible to prevent stress concentration.

(5) Since most of the unseating prevention devices are installed around or near the bearing supports, they should allow for easy inspection and maintenance of the bearing supports. In particular, any structure incorporating protrusions on the superstructure and/or the substructure or connecting the superstructure with the substructure shall be installed in such a way so as not to interfere with the inspection and maintenance, including inspection of the expansion joint or bearing supports. Also, it would be useful to have a slope on the top of the substructure to enhance water discharge.



(a) Example of steel superstructure



(b) Example of concrete superstructure

Figure C7.3-3 Examples of Unseating Prevention Structures Connecting Two Adjacent Superstructures

7.4 STRUCTURE OR DEVICE TO PREVENT SUPERSTRUCTURE FROM SETTLING

The device or structure to prevent the superstructure from settling shall be capable of keeping the superstructure at an appropriate height, even when the bearing supports are damaged.

Commentary C7.4

It is important to keep any gaps on the road surface as small as possible after an earthquake with a large magnitude when the bearing support system is damaged, so that residents can proceed with an emergency evacuation and that the emergency vehicles can pass as soon as possible. For this purpose, structures or devices to protect the superstructure from settling shall be installed, especially for bridges with tall bearings. However, care shall be taken when placing these devices so that it will not interfere with the inspection and maintenance of bearings, expansion joints and other accessories.

Some examples of this device include reserve rubber bearings or concrete block pedestals. Although there is not much effect to the movement of emergency vehicles just after the disaster, it would be better to specify a limit for each route such that the settlement of the road surface after an earthquake is no more than 50~100 mm. It would be better to use unseating prevention devices or structures limiting excessive displacement as structures to prevent the superstructure from settling rather than installing other structures.

In this case, either the structures are capable of supporting the superstructure after the bearings are damaged, or foundations for jack operation are installed for repair or maintenance purposes. This can also avoid complications on top of the pier.

There is no need to consider the horizontal design seismic forces when structures or device for preventing the superstructure from settling can resist the vertical load from the superstructure.

7.5 STRUCTURE OR DEVICE TO LIMIT EXCESSIVE DISPLACEMENT

(1) Structures or devices limiting excessive displacement working in the direction perpendicular to the bridge axis shall be installed at the end supports, in addition to the unseating prevention system working in the bridge axis direction, for the following bridges:

1) Skew bridges with a small skew angle satisfying Equation 7.5-1.

$$\sin 2\theta/2 > b/L \quad \text{.......... (7.5-1)}$$

where:

L : Length of continuous superstructure, (m).

b : Total width of the superstructure, (m).

θ : Skew angle, (degree).

2) Curve bridges satisfying Equation 7.5-2.

$$\frac{115}{\phi}\frac{(1-\cos\phi)}{(1+\cos\phi)} > b/L \quad \text{.......... (7.5-2)}$$

where:

L : Length of continuous superstructure, (m).

b : Total width of the superstructure, (m).

θ : Intersection angle, (degree).

3) Bridges in which the substructure is narrow at the top.

4) Bridges with a small number of bearing supports on one bearing line.

5) Bridges which could be subjected to movement of the bridge piers in the direction perpendicular to the bridge axis as a result of lateral spreading specified in Article 6.3.

(2) Bridges specified in 3), 4) and 5) of the Article (1) above shall have structures limiting excessive displacement installed at intermediate supports.

(3) Structures limiting excessive displacement in the transverse direction to the bridge axis shall be designed so that the sectional forces due to design seismic forces obtained from Equation 7.5-3 do not exceed its factored resistance (ultimate strength).

$$H_S = 3k_h R_d \quad \text{.......... (7.5-3)}$$

where:

H_S : Design seismic force used in the design of the structure or device limiting excessive displacement, (kN).

k_h : Design horizontal seismic force coefficient of Level 1 Earthquake Ground Motion, equivalent to the elastic seismic response coefficient C_{sm} stipulated in Article 3.6.2. However, k_h shall not be greater than 0.20, 0.25 and 0.30 for Ground Types I, II and III respectively.

R_d : Dead load reaction, (kN).

(4) Allowable length of a structure or device limiting excessive displacement shall be greater than its design value. The design value shall approximately be equal to the deformation limit of the bearing support in the bridge axis.

The structure limiting excessive displacement may also have the function of a joint protector, and in this case the design allowance length can be taken as the movement of the bearing support. However, the allowance length of the structure limiting excessive displacement shall not be greater than the allowable expansion length of the expansion joint.

(5) Structures limiting excessive displacement shall not interfere with the functions of the bearing support, including translational and rotational movements of the support.

(6) Structures limiting excessive displacement shall allow easy maintenance and inspection of the bearing support.

(7) The location of structures limiting excessive displacement shall not affect the functions of an unseating prevention structures specified under Article 7.3 of this Section.

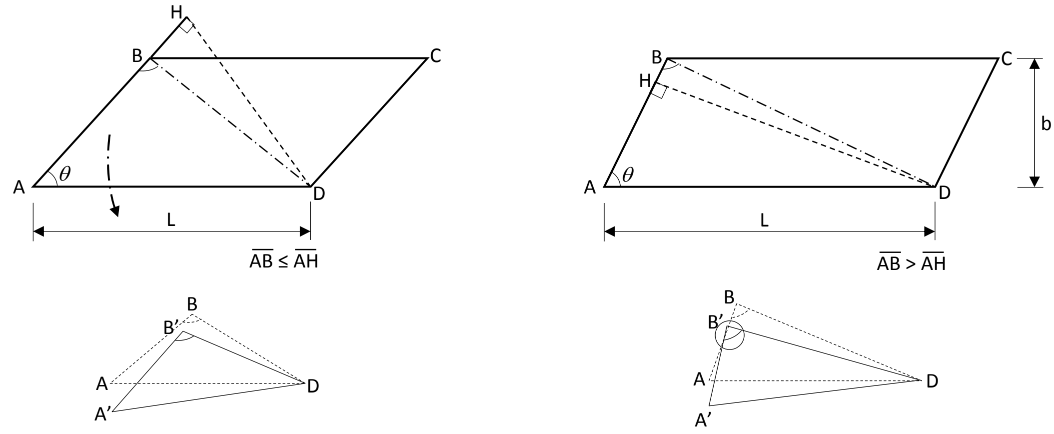
Commentary C7.5

(1) Since unseating of a bridge may hardly happen due to the movement of the girder in the transverse direction to the bridge axis when Type B bearings are used, there is no need to install an unseating prevention system in this direction. However, as indicated in the commentary for Article 7.1, structures or devices limiting excessive displacement should be installed even when Type B bearings are used in cases such as:

- bridges such as skew bridges,
- curved bridges,
- bridges in which the substructure is narrow at the top,
- bridges with fewer bearings on one bearing line, or
- bridges in which unseating due to movement transverse to the bridge axis may happen because the columns may move in this direction under the effects of lateral spreading.

However, there is no need to install additional structure limiting excessive displacement when Type A bearing supports are used since the structure limiting excessive displacement is installed to supplement the Type A bearing supports.

Equations 7.5-1 and 7.5-2 indicate a condition that the superstructure can rotate without being restricted by the adjoining girders or the abutment parapets, depending on the geometric conditions of the superstructure, as shown in Figures C7.5-1 to C7.5-3. From the characteristics of the restraints, whether it is known that the superstructure may rotate or not is little affected by the portions adjoining the two ends of the superstructure without being restricted by girders or abutment parapets. Figures C7.5-4 to C7.5-5 show the relations of Equations 7.5-1 and 7.5-2, respectively.



(a) In case ∠DBA ≥ 90 (can rotate)   (b) In case ∠DBA < 90 (can rotate)

Figure C7.5-1 Conditions in Which a Skew Bridge can Rotate without Being Affected by the Adjoining Girders or Abutment

(2) For bridges such as skew bridges, curved bridges, bridges in which the substructure is narrow at the top, bridges with fewer bearings on one bearing line, or bridges in which unseating due to movement transverse to the bridge axis may happen because columns may move in both longitudinal and transverse directions under the effects of lateral spreading, the structure or device limiting excessive displacement should be installed at intermediate supports in addition to end supports.

(3) With the same function as the structure limiting excessive displacement in the bridge longitudinal axis direction, a similar structure in the transverse direction to the bridge axis shall be designed in accordance with Article 7.5. The design allowance length of structure limiting excessive displacement in the transverse direction to the bridge axis shall be the amount of movement of the bearing support for Level 2 Earthquake Ground Motion. Expansion gap shall not restrict any relative movement of the superstructure and the substructure in the same direction, while movement in this direction caused by temperature changes need not be considered.

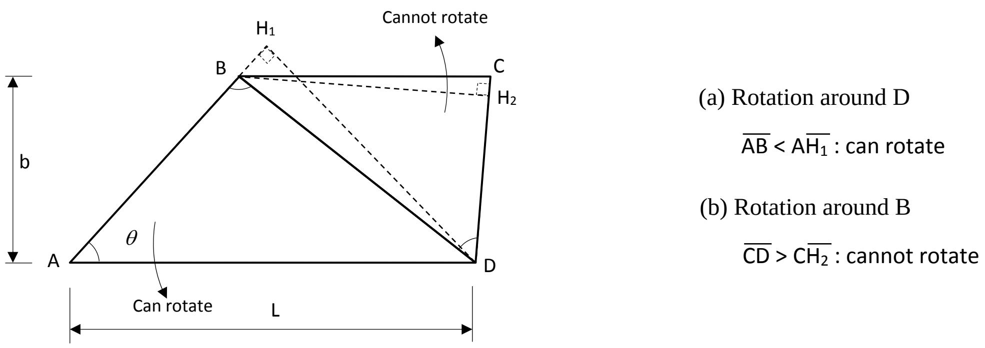


Figure C7.5-2 Conditions in which a Skew Bridge with Unparallel Bearing Lines on Both Edges of the Superstructure can Rotate.

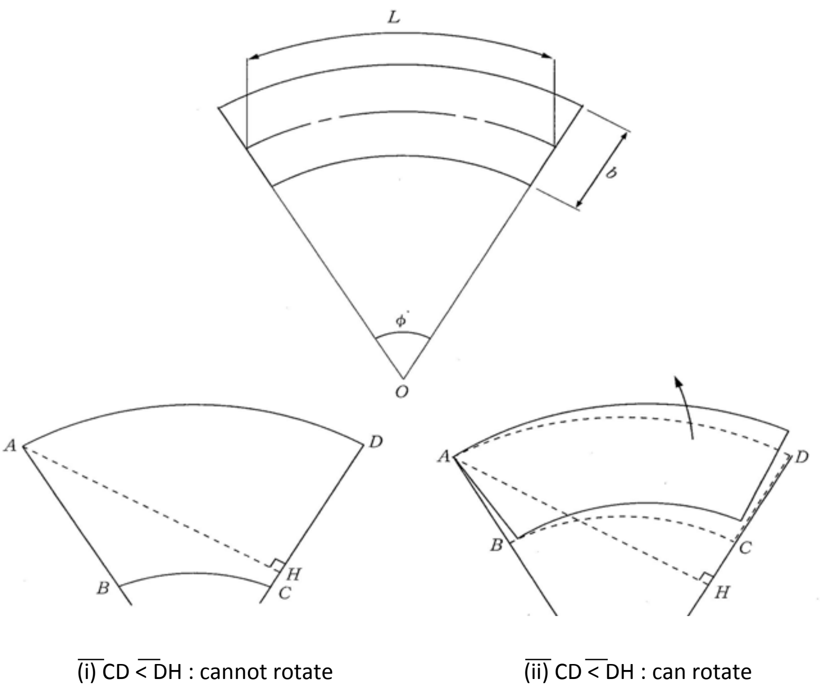


Figure C7.5-3 Conditions in which a Curved Bridge can Rotate Without Being Affected by Adjoining Girders or Abutment

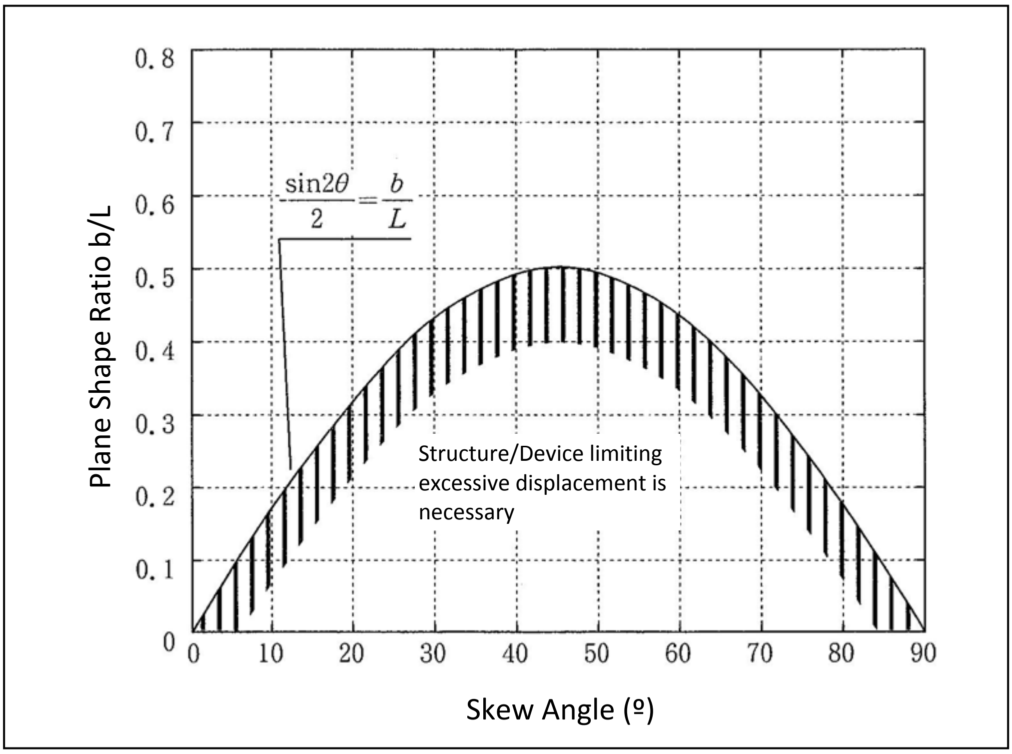


Figure C7.5-4 Conditions in which a Skew Bridge Requires a Structure Limiting Excessive Displacement in the Transverse Direction to the Bridge Axis

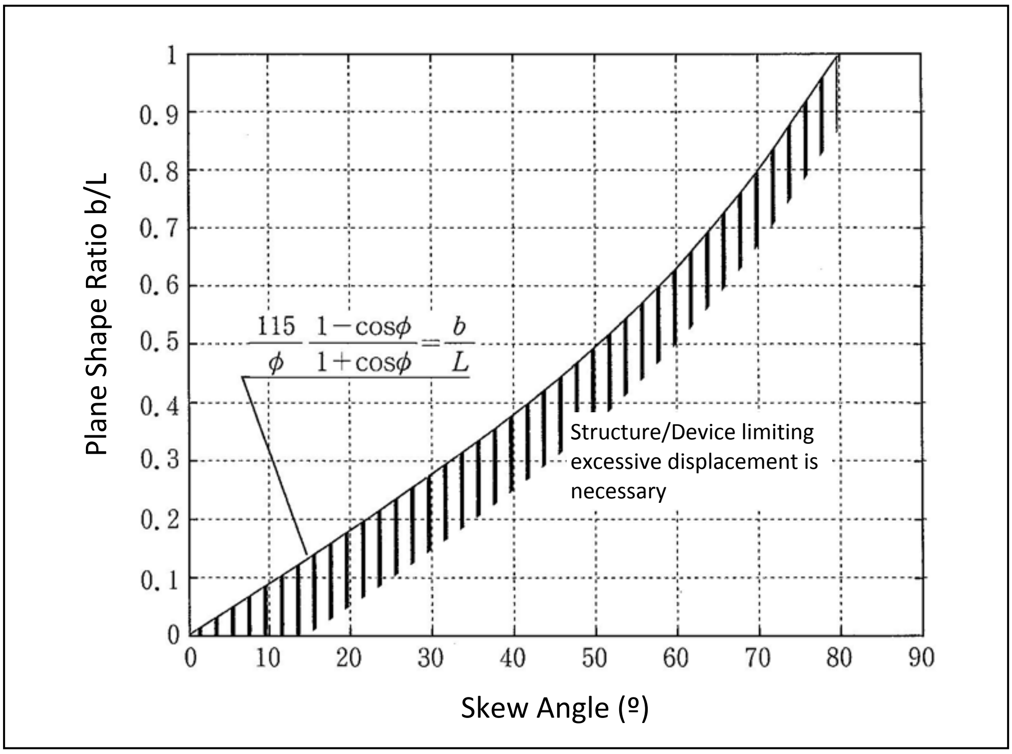


Figure C7.5-5 Conditions in which a Curved Bridge Requires a Structure Limiting Excessive Displacement in the Transverse Direction to the Bridge Axis

A structure or device limiting excessive displacement can be in the form of a structure connecting the superstructure and substructure or a structure with a protrusion designed in the superstructure and substructure. An example of a structure limiting excessive displacement is shown in Figure C7.5-6. The actual structural forms of a structure or device limiting excessive displacement includes a structure limiting the relative displacement between the superstructure and substructure by means of an anchor bar or a steel angle stopper or a concrete structure (such as shear key) installed on the top of the substructure or in the superstructure.

A structure or device limiting excessive displacement is designed for the purpose of working together with the bearing support to resist the seismic forces generated due to Level 2 Earthquake Ground Motion. Considering that the ultimate strength equivalent to that of Type B bearing support shall be guaranteed, the design seismic force to be used in the design of the structure or device limiting excessive displacement is provided in Equation 7.5-3. Here, during the calculation of the design seismic force used for the design of the structure limiting excessive displacement, although the design horizontal seismic coefficient correspond to the design horizontal seismic coefficient of Level 1 Earthquake Ground Motion is being adopted, the design seismic force shall be determined by Equation 7.5-3 in order to guarantee the ultimate strength equivalent to that of Type B bearing support.

It is necessary to design the joint section of the structure limiting excessive displacement into a structure capable of transmitting the design seismic forces to the superstructure and substructure. For this purpose, the design seismic force given by Equation 7.5-3 shall be considered for the joint connection of the structure limiting excessive displacement, and its safety shall be verified by following the relevant specifications corresponding to Steel, Concrete and Substructures of the *AASHTO LRFD Bridge Design Specifications (2012 or later).*

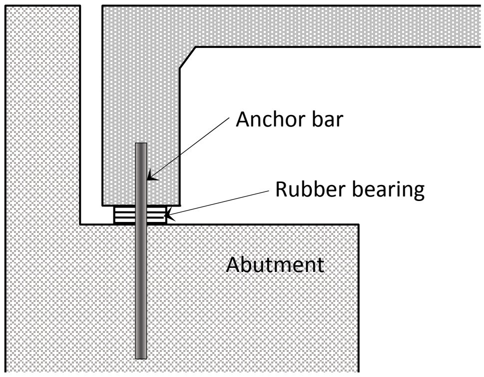


Figure C7.5-6 Details of Structure or Device Limiting Excessive Displacement Connecting Superstructure and Substructure

(4) It is stipulated that the gap distance or movement allowed by the structure limiting excessive displacement shall satisfy Equation C7.5-1. In the event that the bearing support suffers from damage, it is necessary to enable the structure limiting excessive displacement to function immediately to prevent the relative displacement between the superstructure and substructure from becoming excessively large. Therefore, the design gap distance of the structure limiting excessive displacement shall be set to be about the same as the deformation capacity of the bearing support. However, it is recommended to keep an allowance for the design gap distance to mitigate the margin of error in installation. The recommended design gap distance is given in Equation C7.5-2. However, if this allowance is set to be excessively large, a deformation occurring beyond the deformation capacity of the bearing could hinder the structure limiting excessive displacement to perform its original function. In this regard, an unnecessarily large allowance for the gap distance shall be avoided. Generally, the standard allowance of the gap distance of the structure limiting excessive displacement, can be taken to be about 15mm in the positive and negative directions.

$$L_S \geq L_{Sd} \qquad \text{(C7.5-1)}$$

$$L_{Sd} = L_E + L_A \qquad \text{(C7.5-2)}$$

where:

L_S : Gap distance of the structure limiting excessive displacement, (mm).

L_{Sd} : Design gap distance of the structure limiting excessive displacement, (mm).

L_E : Distance of translational movement under Level 1 Earthquake Ground Motion. In case of a rubber bearing, it is the displacement equivalent to the allowable shear strain of the rubber, (mm).

L_A : Allowance for the gap distance of the structure limiting excessive displacement, (mm). Can be taken as 15mm.

In addition, the structure or device limiting excessive displacement can function as a joint protector. In case the design gap distance is regarded as the distance of the translational movement of a bearing support, the gap distance can be designed to satisfy Equation C7.5-1. However, the function of the joint protector can no longer be expected when the gap distance exceeds the allowable expansion length of the expansion joint. Therefore, the gap distance shall not exceed the allowable expansion length of the expansion joint.

(5) The structure limiting excessive displacement shall not interfere with the functions of the bearing support for translational and rotational movements, which are brought about by the temperature changes or live loads of the superstructure.

(6) Since the structure limiting excessive displacement is usually installed near the bearing support, it shall be designed to avoid being an obstacle to the maintenance work such as inspection or repair of the bearing support. Also, proper drainage at the top of the substructure shall be ensured through proper slope of the top surface of the substructure.

(7) The structure or device limiting excessive displacement and the unseating prevention device or structure are required to play different roles during an earthquake. It should be noted that the deformations and timing to start their functions differ from each other. Moreover, there is a potential risk that the fail-safe function of the unseating prevention system will not perform effectively when they are installed in one concentrated location. Therefore, the installation position of the structure limiting excessive displacement shall be carefully considered in order to avoid any interference with the functions of the unseating prevention system.

SECTION 8:

REQUIREMENTS FOR SEISMICALLY ISOLATED BRIDGES

8.1 GENERAL

(1) The introduction of seismic isolation design shall be considered to increase the natural period of vibration, such that the bridge is subjected to lower earthquake forces, as well as to increase the energy absorption capacity of the bridge for both normal and earthquake conditions. However, seismic isolation shall not normally be adopted for a bridge meeting the following conditions:

1) The bridge is located in a soil layer for which the seismic geotechnical parameter determined from Article 6.2.4 of these Specifications is zero.

2) The bridge has fairly flexible substructures and long fundamental natural period.

3) The bridge is located in a soft soil layer with long natural period that may cause resonance with the bridge if seismic isolation is introduced.

4) The bridge has uplift at bearing supports.

(2) When seismic isolation is adopted, the bridge should be designed with emphasis on enhancement of damping properties using a high energy absorption system, and on transfer of seismic forces to substructures. Excessive increase in the natural period of vibration of the bridge shall be avoided.

(3) The natural period of vibration of a seismically-isolated bridge shall be determined in such a manner that the energy is absorbed by the isolation bearings and that an increase in seismic displacement of the superstructure does not cause any adverse effect on the bridge functions.

(4) Seismic isolation bearings with good proven performance and simple mechanisms shall be selected and adopted within the range of acceptable mechanical behavior. Seismic isolation bearings shall be securely fixed to the superstructure and substructure by anchor bolts and shall be capable of replacement.

(5) When seismic isolation is adopted, sufficient allowance for gaps and spaces between main structures such as abutments, piers and girders shall be kept so that the displacement assumed in the design of an isolation system is possible.

(6) When a seismic isolation bearing is used as a structure or device to distribute horizontal forces to the substructures during an earthquake, rather than a seismic force reduction structure through energy absorption, the design structural response shall not be reduced based on the damping ratio of the bridge.

Commentary C8.1

(1) It is suggested that the natural period of vibration of an isolation bearing should be lengthened to an appropriate extent so that improvement of damping performance and reduction of the superstructure inertia force during an earthquake can be expected. A seismically-isolated bridge, while being a structure distributing seismic horizontal forces to disperse the superstructure inertia force to multiple substructure, is characterized by its capability of reducing the superstructure inertia force during an earthquake by way of lengthening the natural period and improving the damping performance through isolation bearings. It is thus necessary to get a thorough understanding of the inherent characteristics of

a seismically-isolated bridge when deciding whether to adopt seismic isolation design or when analyzing the seismic behavior of a seismically-isolated bridge.

When seismic isolation system is adopted to mitigate earthquake effects for bridges, the structure shall satisfy all of the following conditions:

1) The seismic isolation device, in addition to the vertical load-carrying capacity, should provide lateral flexibility to lengthen the natural vibration period and reduce seismic forces in the event of large earthquakes.

2) Since over-dependence on the lengthening of the natural vibration period will cause an excessive displacement in the bridge during an earthquake, a damper or energy dissipator is also introduced to limit the displacement to a practical design level so that no problem will occur during service state.

3) The isolator or the damper adopted should provide the necessary rigidity under low (service) load level to prevent the vibration from affecting the bridge serviceability caused by the wind or braking load etc.,.

An isolator is a device that has the flexibility to support the superstructure in the lateral direction while carrying its weight at the same time. A laminated rubber bearing is primarily used as an isolator. A damper, on the other hand, is a device that absorbs energy based on the theories of hysteretic damping and viscous damping to improve the damping performance of the bridge.

Here, only the structures that meet the above three conditions shall be applicable, for example, a rubber-base isolation bearing unit containing both isolator and damper just like a laminated bearing with a built-in lead plug or a high damping laminated rubber bearing, which are increasingly used in practice. In addition to these, new technologies such as active control or semi-active control are being developed as described in Article 8.5 of this Section.

Using an isolation bearing improves the deformation characteristics and energy absorption performance of the bridge and also enables application of multi-span continuous bridges and bridge structures without joints. In view of this, it is desirable that the continuity of the superstructures should be included when adopting a seismically-isolated bridge. However, the suitability of adopting a seismically-isolated bridge depends on the structural conditions of the bridge and the conditions of the ground around the foundation. Moreover, the seismic performance of a bridge is greatly affected by the stability of the ground around the foundation during an earthquake.

A seismically-isolated bridge is designed to provide a flexible connection between the superstructure and the substructure or between the foundation and the pier body that will allow for the relative displacements to occur between these members which in turn will reduce the seismic force acting on the bridge. For this reason, when the ground around the bridge foundation is found to be unstable during an earthquake, the seismic isolation effects anticipated in the design may not be achieved. In this case, a seismically-isolated bridge shall not be adopted. Sometimes, the capacity of the ground around the foundation can be severely affected by the unstable ground during an earthquake as described in Section 6 of these Specifications. For this reason, it is a rule that when a soil layer exists with its geotechnical parameter specified in Article 6.2.4 of these Specifications being zero in seismic design, a seismically-isolated bridge shall not be adopted.

In addition, when a bridge has tall piers or with a special configuration with flexible substructures and a long natural period, it is easy for the substructures to suffer from significant displacement during a large earthquake thus limiting the reduction of inertia forces due to the long natural period. In this case, it is stipulated that a seismically-isolated bridge must not be adopted. Here, the standard for judging the long natural period of a bridge, though depending on the scale and type of the bridge, can generally be set as roughly above 1.0 second on the assumption that all the supporting conditions are fixed. Therefore, when seismic isolation design is adopted for a bridge with a natural period longer than this, it is recommended that careful study shall be made on problems such as the displacement of the substructure during an earthquake, the reduction effect of inertia force due to lengthening of the natural vibration period and high damping.

Moreover, careful attention shall be taken in order to ensure that the resonance between the ground and the bridge does not occur due to the adoption of a seismically-isolated bridge. When considering the possibility of resonance between the ground and the bridge, measures shall be made to avoid the closeness between the values of the natural periods of the ground and the bridge during an earthquake. Equation 3.5.1-1 shall be used as the basis in determining the natural period of the ground which takes into account the stiffness of the ground equivalent to the strain occurring on the ground during an earthquake. Moreover, it is better to examine the natural vibration characteristics of the surface ground when necessary.

The dynamic characteristics such as the ultimate strength and energy absorption performance of the bearing are yet to be confirmed when the seismic isolation bearing is subjected to earthquake forces in the horizontal direction together with uplift or negative vertical reaction. In this regard, it is stipulated that a seismic isolation system must not be adopted when the bearing is anticipated to have a negative vertical reaction or uplift. Here, the case in which a negative vertical reaction occurs refers to the state when the vertical reaction of the bearing with the dead load being taken into account becomes negative.

In the previous discussions, the cases where seismically-isolated bridge shall not be adopted have been specified. On the other hand, the cases which are suitable for providing a seismically-isolated bridge are as follows:

1) When the ground is firm and the ground around the bridge foundation is stable during an earthquake.

2) When the stiffness of the substructure is high, and the natural period of the bridge is short

3) When it is a multi-span continuous bridge.

In the above conditions, reduction of earthquake forces acting on the bridge can very well be expected through lengthening of the natural vibration period of the structure. However, since the effect of seismic isolation design does not generally work well on simple girder bridges, a multi-span continuous bridge is preferred for seismic isolation which enables distribution of earthquake forces to all the substructures supporting the bridge. Although continuous multi-span bridges increases the displacement of the girder during service condition by factors like change in temperature, such displacement can be absorbed by the isolation bearing. The displacement occurring on the girder during an earthquake can also be reduced more effectively compared to ordinary rubber bearing.

For the structure and design of an isolation bearing, the following references are recommended:

- *Design Manual of Bearings for Highway Bridge*, Japan Road Association, and
- *Guide Specifications for Seismic Isolation Design, 3rd ed. July 2010,* American Association of State Highway and Transportation Officials.

(2) A seismically-isolated bridge is built to reduce the inertia force by prolonging natural period and enhancing the damping performance of the bridge. In some countries, there are examples of reducing the design seismic forces by applying fairly short natural periods. However, in other countries, ground conditions are generally rather soft and large scale earthquakes tend to occur where the seismic ground motion component with a long natural period easily becomes dominant. Under these conditions, when determining the design horizontal seismic coefficient for the Level 2 Earthquake Ground Motion, careful attention should be given the periods (for Types I – III ground profiles) when the values of the coefficients begin to reduce. For this reason, unless a longer natural period than these natural periods is created, the reduction effect of the inertia force will be limited.

However, a longer natural period does not only mean a reduction of the superstructure inertia force transmitted to the substructure, but also mean an increase in the displacement of the superstructure during an earthquake. Therefore, reducing the inertia force by creating a longer natural period may prove to be difficult for Type III ground. In this case, improvement of the damping performance shall be the focus of seismic isolation design. In the effort of reducing the inertia force by creating a longer natural period in addition to improving the damping performance, careful consideration shall be given to restricting the increase in displacement of the superstructure during an earthquake within the allowable scale in design. An excessively long natural period shall be avoided.

(3) The natural period of a bridge with a seismic isolation bearing shall be determined in such a way that while ensuring effective absorption of energy by the isolation bearing, no adverse effect on the function of the bridge due to an increase in the displacement of the superstructure during an earthquake shall be allowed. In order to achieve this, a general standard for the natural period of a bridge with isolation bearings can be set as *more than 2 times the natural period* of a bridge without isolation bearings. If the difference between the natural period of the superstructure and that of the substructure is about this much, the influence of the coupled vibrations of the superstructure and substructure can be kept small, and the displacements of the bridge will mainly be concentrated in the isolation bearing. On the contrary, if the natural period of a bridge with an isolation bearing is less than 2 times the natural period of a bridge without any isolation bearing, the deformations sometimes may be concentrated in the substructure instead of the isolation bearing. In this situation, the isolation bearing will not be able to function effectively. Seismic isolation design shall be done to prevent this from happening. The natural period of a bridge without an isolation bearing can be assumed to be the natural period when all the bearings are regarded as fixed bearings. It should be noted that the standard of more than 2 times the natural period of a bridge with an isolation bearing has been set for verification for Level 2 Earthquake Ground Motion only. For verification for Level 1 Earthquake Ground Motion, it is acceptable that this condition does not have to be satisfied. On the other hand, if the natural period is set as more than 2 times, a reasonable design may turn out to be difficult due to the excessive displacement of the superstructure. In such a case, although it is acceptable to set the increasing rate of the natural period as less than 2 times, careful study shall be made to ensure that the deformations shall be concentrated in the isolation bearing instead of in the substructure.

(4) When selecting a seismic isolation system, the following standards and conditions for the seismic isolation bearing shall be met:

1) It shall be simple in mechanism, but with full function and shall able to be used within the scale where its mechanical behaviors are well understood.

2) It shall demonstrate a stable performance throughout the period while the earthquake ground motion last.

3) In terms of its materials and mechanism, the isolation bearing shall serve for a long period of time in a stable working conditions. Since an isolation bearing may deteriorate in quality or be damaged after an extended service period, it is necessary to have a mechanism that allows replacement of an isolation bearing with the same properties as in the design.

4) Anchor bolts and other fixtures shall have the capacity to effectively transmit the seismic forces between the isolation bearing and the girder or between the bearing and the substructure.

(5) Since creating a longer natural period will increase the displacement of the superstructure during an earthquake, the main concern is to provide sufficient gaps between the abutment and the superstructure, between adjacent superstructures and between major structural members. A seismically-isolated bridge is designed based on the assumption that the displacement equivalent to the design displacement of the isolation bearing will be generated in the bearing. It is therefore necessary to avoid the possibility where the assumed design displacement fails to occur because of the impact between the abutment and the

superstructure. It is suggested that space or gap is designed at the end of the superstructure by following the recommendations of Article 5.8 in principle. However, a large spacing might mean a longer expansion gap at the expansion joint, forcing its structure to become larger. This may cause problems in maintenance, driving conditions, vibration and noise. Therefore, when adopting a seismically-isolated bridge, the following measures are recommended to be examined:

1) A bridge having a multi-span continuous structure to create a structural system that vibrates together as a whole.

2) An expansion joint and an abutment capable of absorbing the large displacement that will possibly occur at the end of the superstructure or the abutment. Measures to ensure that the gap or spacing can truly be maintained during a large earthquake shall be carefully examined.

When two superstructures are designed to share one pier in the bridge, careful attention shall be given to avoid a significant difference between the natural periods of the two design vibration units in order to prevent damaging effects due to collision of the two design vibration units.

(6) An isolation bearing can also be used as a bearing simply for distributing the horizontal forces to the substructures during an earthquake, for which the reduction of seismic force by energy absorption is not expected. In such a case where seismic design is not considered, since lengthening of the natural period and improvement of energy absorption performance of the seismic isolation bearing will not be included in the verification, reduction of the seismic ground motion based on the damping ratio of the bridge shall not be allowed. In addition, if verification of seismic performance for a bridge with this kind of design is carried out by the dynamic method, considerations shall be given when modeling the bearing to prevent too much expectation on the damping effect of the isolation bearing. For this kind of bearing, it is sufficient to use about the same damping ratio with an ordinary rubber bearing.

In addition, when an isolation bearing is used as a bearing for distributing the horizontal forces during an earthquake, the ductility capacity and the allowable displacement of the pier shall be calculated and verified.

8.2 PERFORMANCE VERIFICATION OF SEISMICALLY ISOLATED BRIDGES

(1) The verification for seismic performance of a seismically-isolated bridge shall be done by dynamic analysis methods depending on the bridge properties, structural configurations and complexity of seismic behavior or by static analysis method as approved by DPWH. Bridges without complicated seismic behavior may be analyzed by static method while bridges with complicated seismic behavior shall be analyzed by dynamic method. When calculating the seismic response of a seismically-isolated bridge, the isolation bearing can be modeled in accordance with the provisions given in Article 8.3 of this Section.

(2) The allowable ductility ratio of reinforced concrete columns in a seismically-isolated bridge shall be obtained from Equation 8.2-1.

$$\mu_m = 1 + \frac{\delta_u - \delta_y}{\alpha_m \delta_y} \qquad \text{(8.2-1)}$$

where:

μ_m : Allowable ductility ratio of reinforced concrete columns in seismically-isolated bridge.

α_m : Safety factor used for the calculation of the reinforced concrete columns. α_m shall be calculated by Equation 8.2-2.

$$\alpha_m = 2\alpha \quad \text{(8.2-2)}$$

α : Safety factor used for calculation of the allowable ductility ratio of the reinforced concrete columns, and is specified in Table 8.2-1.

Table 8.2.1 Safety Factor α for Calculating Ductility Capacity of a Reinforced Concrete Column Resulting in Flexural Failure

Seismic Performance to be Verified	Safety Factor α for Calculating Ductility Capacity	
	Plate Boundary Type Earthquake Ground Motion (Type I)	Inland Direct Strike Type Earthquake Ground Motion (Type II)
Seismic Performance Level 2 (SPL-2)	3.0	1.5
Seismic Performance Level 3 (SPL-3)	2.4	1.2

δ_y, δ_u : Yield displacement and ultimate displacement of the reinforced concrete columns, respectively.

(3) Isolation bearings shall be verified in accordance with the provisions in Article 5.7. In addition, only isolation bearings having the fundamental functions specified in Article 8.4 of this Section shall be basically selected.

(4) Design of superstructure end of a seismically-isolated bridge shall be based on Article 5.8 of these Specifications.

Commentary 8.2

(1) Although verification of the seismic performance of a seismically-isolated bridge shall be done based on Article 3.3, proper considerations of the bridge properties shall be included in the verification. For a seismically-isolated bridge subjected to Level 2 Earthquake Ground Motion, the nonlinearity of the isolation bearing and the secondary plasticity of the pier shall be taken into account. A seismically-isolated bridge shall be classified as a bridge having complex seismic behaviors during an earthquake and verification of such bridge shall be carried out by dynamic analysis method. During the verification process by dynamic analysis method, the dynamic analysis procedures shall be utilized while the items listed below shall be confirmed at the same time.

1) The displacement shall be absorbed mainly by the isolation bearings and no excessive displacement shall be concentrated in the substructure.

2) The damping performance of the bridge shall be improved by the isolation bearings.

Moreover, for a typical seismically-isolated bridge, once the first mode which basically consists of the displacement of the isolation bearing becomes dominant, the vibration characteristics of all the piers will turn out to be roughly the same in most cases. Therefore, during the process of modeling, unnecessarily complicated models should be avoided. Each pier can be separated, when necessary, in order to create a model that is capable of appropriately reflecting the vibration characteristics of the bridge.

(2) When seismic isolation design is adopted, the safety factor used for calculating the ductility capacity of a reinforced concrete column shall be twice the values shown in Table 8.2-1.

The purpose of this will be to restrict the response occurring in a reinforced concrete column as a secondary plastic deformation and to minimize damage. At the same time the tasks of lengthening the natural period and absorbing energy shall be done by the isolation bearing and not by the pier.

Here, the ductility capacity of a steel pier is not provided. The reason for this is that since bridges with steel piers generally have long natural periods it is not common and reasonable to adopt seismic isolation design for such a bridges. In case that seismic isolation design is introduced for a bridge with steel piers, careful attention is needed to confirm if it is possible for the isolation bearing to achieve lengthening of the natural period and absorbing the energy.

(3) The design seismic force used for verification of an isolation bearing and the verification method shall be in accordance with the provisions specified in Article 5.7 of these Specifications. As stated in Article 5.7.1, an isolation bearing shall be classified as a Type B support and its verification shall be performed for Level 2 Earthquake Ground Motion. This is because in a seismically-isolated bridge, the isolation bearing is an important structural element for which reduction of inertia force is expected during an earthquake and in verification for Level 2 Earthquake Ground Motion, the property of an isolation bearing plays a dominant role.

In addition, as a principle, in order for an isolation bearing to function properly under the assumed design conditions, an isolation bearing possessing the basic properties and functions specified in Article 8.4 shall be adopted.

(4) When adopting a seismically-isolated bridge, it is proper to assure that the impact between the end of the superstructure and the abutment or the end of an adjacent superstructure will not occur so that the assumed displacement in design can be attained in the isolation bearing. Verifications shall be conducted and considerations be given to the structure of the end of a superstructure based on Article 5.8 of these Specifications.

8.3 ANALYTICAL MODEL OF ISOLATION BEARINGS

8.3.1 General

An isolation bearing can be modeled as a nonlinear member having inelastic hysteretic property or an equivalent linear member having equivalent stiffness and damping ratio. In the case of modeling an isolation bearing as a nonlinear member, reference shall be made to provisions in Article 8.3.2 of this Section. In the case of modeling an isolation bearing as an equivalent linear member, design displacement and equivalent stiffness and damping ratio of the bearing shall be based on Article 8.3.3 of this Section.

Commentary 8.3.1

Since an isolation bearing generally has inelastic hysteretic property, it is required that it should be modeled into a nonlinear member or an equivalent linear member capable of expressing such characteristics appropriately. In case of verification of seismic performance based on the static method, an isolation bearing can be modeled by following the equivalent linear method. In the equivalent linear method, the properties of a member having inelastic hysteretic property are modeled into a linear member having an equivalent stiffness and an equivalent damping ratio. During application of this method, the displacement that may occur in the isolation bearing shall first be assumed before the analysis. Then, the equivalent stiffness and the equivalent damping ratio correspondent to the assumed displacement shall be obtained. After calculating the displacement occurring in the isolation bearing in that configuration, repeated calculations of the value thus obtained shall be done until it converges into the value of the displacement of the isolation bearing which has been assumed at the beginning of design.

A laminated rubber bearing with a built-in lead plug and a high damping laminated bearing are commonly used as isolation bearings. Their inelastic hysteretic property can be found by referring to the *Handbook of Bearing for Highway Bridge (Japan Road Association)* or other reference materials for bearings, as approved by DPWH. In addition, the *Guide Specifications for Seismic Isolation Design, 3rd ed. July 2010*, American Association of State Highway and Transportation Officials provides some requirements for the design properties of seismic isolation system and elastomeric bearings.

8.3.2 Inelastic Hysteretic Model of Isolation Bearings

In modeling an isolation bearing as a nonlinear member, a bilinear model illustrated in Figure 8.3.2-1 shall be applied. In this case, the initial stiffness and secondary stiffness shall be appropriately determined according to its properties.

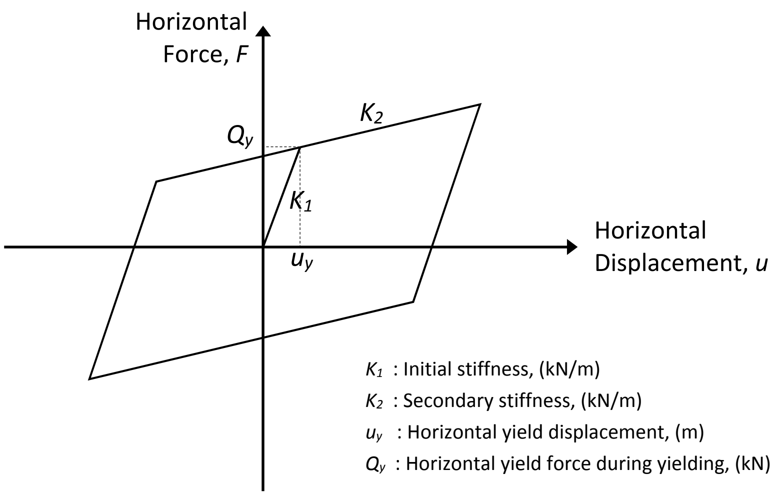


Figure 8.3.2-1 Inelastic Hysteretic Model of Isolation Bearing

Commentary 8.3.2

Since an isolation bearing generally has inelastic hysteretic property, it is necessary to use a model capable of expressing appropriately the isolation bearing properties and behavior during the time history response analysis for verification of a seismically-isolated bridge. As stated in this provision, the laminated rubber bearing with a built-in lead plug and the high damping laminated rubber bearing used commonly can generally be expressed as a bilinear model. In this case, however, it is required that the isolation bearing shall be given an appropriate stiffness corresponding to its characteristics. However, if an isolation bearing cannot be expressed as a bilinear model, its properties shall be modeled appropriately.

8.3.3 Equivalent Linear Model of Isolation Bearings

When an equivalent linear member is used to model an isolation bearing, the design displacement, and the equivalent stiffness and damping ratio shall be obtained as follows:

(1) Design displacement of an isolation bearing

To calculate the equivalent stiffness and damping ratio of an isolation bearing, design displacement (u_B) and the effective design displacement (u_{Be}) can be obtained from Equations (8.3.3-1) and (8.3.3-2).

$$u_B = \begin{cases} \dfrac{k_h W_u}{K_B} & \textit{(in case of verification for Level 1 Earthquake Ground} \\ \dfrac{c_m P_u}{K_B} & \textit{(in case of verification for Level 2 Earthquake Ground} \end{cases} \quad \text{(8.3.3-1)}$$

$$u_{Be} = c_B u_B \quad \text{(8.3.3-2)}$$

where:

u_B : Design displacement of an isolation bearing, (m).

u_{Be} : Effective design displacement of an isolation bearing, (m).

k_h : Design horizontal seismic force coefficient of Level 1 Earthquake Ground Motion, equivalent to the elastic seismic response coefficient C_{sm} stipulated in Article 3.6.2. However, k_h shall not be greater than 0.20, 0.25 and 0.30 for Ground Types I, II and III respectively.

P_u : Horizontal force equal to the lateral strength of a pier when considering plastic behavior of the pier (plastic shear), or the horizontal force corresponding to the maximum response displacement of a foundation within its plastic behavior, (kN).

c_m : Dynamic modification factor used for the calculation of design displacement of an isolation bearing, and can be taken as 1.20.

K_B : Equivalent stiffness of an isolation bearing used for the calculation of u_B, (kN/m).

W_u : Weight of superstructure, whose horizontal force is borne by the isolation bearing, (kN).

c_B : Modification factor representing the dynamic properties of the inertia force and shall be taken as 0.70.

(2) Equivalent stiffness and damping ratio of an isolation bearing

The equivalent stiffness and damping ratio of an isolation bearing can be calculated from Equations (8.3.3-3) and (8.3.3-4) respectively, as follows:

$$K_B = \frac{F(u_{Be}) - F(-u_{Be})}{2u_{Be}} \quad \text{(8.3.3-3)}$$

$$h_B = \frac{\Delta W}{2\pi W} \quad \text{(8.3.3-4)}$$

where:

K_B : Equivalent stiffness of an isolation bearing, (kN/m).

h_B : Equivalent damping ratio of an isolation bearing.

$F(u)$: Horizontal force necessary to produce the horizontal displacement u of an isolation bearing, (kN).

W : Elastic energy of an isolation bearing, equal to the triangular area in Figure 8.3.3-1, (kN·m).

ΔW : Total energy absorbed by an isolation bearing, and equal to the area inside the hysteretic curve of horizontal displacement-force relationship, shown in Figure 8.3.3-1, (kN·m).

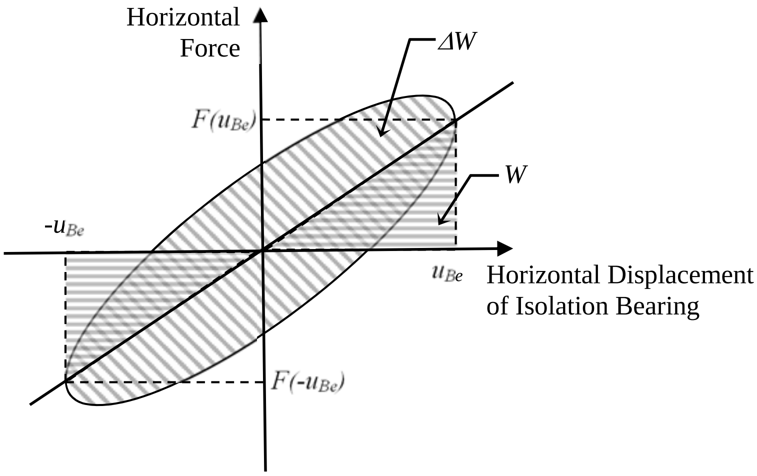


Figure 8.3.3-1 Equivalent Stiffness and Damping Ratio of an Isolation Bearing

Commentary 8.3.3

(1) When modeling an isolation bearing into an equivalent linear member, the displacement that is likely to occur to the bearing shall be assumed. During verification for Level 1 Earthquake Ground Motion, the displacement occurring on the isolation bearing can be taken as the design displacement, u_B, when the inertia force obtained by using the design horizontal seismic coefficient specified in Level 1 design response spectrum coefficient of Article 3.6.2 of this Specifications is used. However, when verification for Level 2 Earthquake Ground Motion, if pier plasticity is being considered, the horizontal force equivalent to the ultimate strength of the pier or the pier plastic shear force can be used. And, either of these horizontal forces shall be multiplied by the dynamic modification factor used for calculation of the design displacement of the isolation bearing. When the product is obtained, (which is the horizontal force acting on the bridge), the displacement occurring in the bearing can be taken as the design displacement u_B.

In case that plasticity is considered for the foundation or the pier, since the design seismic force acting on the bearing generally remains constant, there is no need to do repeated calculations. However, if plasticity is not considered for the foundation nor for the pier, (due to the inelastic hysteretic property of the isolation bearing), the stiffness or damping ratio will generally vary according to the design displacement of the isolation bearing. Therefore, the design displacement of the isolation bearing shall be based on the equivalent stiffness and the equivalent damping ratio applied to the equivalent linear model which shall be used to calculate the seismic response of the bridge. The analysis results on the displacement obtained for the isolation bearing and the damping ratio shall be adjusted with the design displacement and damping ratio assumed at the beginning of the design for the isolation bearing. In case that the design displacement assumed for the isolation bearing by using Equation 8.3.3-1 differs from the displacement obtained for the isolation bearing obtained from the above process, the design displacement of the isolation bearing has to be revised and the same calculations shall again be done. Although it is hoped that the resulting displacement in the isolation bearing obtained from the seismic isolation design can be as close as possible to the design displacement of the isolation bearing, it would be unreasonable to perform excessive calculations for this purpose. Therefore, in order to prevent a large difference in the equivalent stiffness from affecting the natural period and the seismic response during an earthquake, it is recommended that the displacement obtained in the isolation bearing calculated in the isolation design in generally restricted within ±10% of the design displacement of the isolation bearing.

In some cases, due to the difference between the phases of the superstructure and the piers, the displacement of the isolation bearing, which is obtained when a static horizontal force equivalent to the ultimate strength of the pier (plastic shear forces) or the horizontal force equivalent to the maximum response displacement of the foundation is applied, may be smaller than the displacement actually occurring in the isolation bearing. To make up for this difference, the dynamic modification factor c_m used for calculation of the design displacement of the isolation bearing is designated as 1.2 based on a comparison between the results of the static and dynamic analyses.

The displacement response with respect to time occurring in an isolation bearing during an earthquake may vary, according to the characteristics of the earthquake ground motion. But, in order to determine the equivalent stiffness and equivalent damping ratio of the isolation bearing, the value corresponding to the actual response (the effective value) shall be more important than the value corresponding to the maximum displacement occurring in the isolation bearing. Here, the actual value shall be called the effective design displacement of the isolation bearing and shall be obtained from Equation 8.3.3-2. The modification factor c_B which expresses the dynamic properties of the inertia force has been set as 0.70. This is because in order to reflect the seismic response of a bridge by using the equivalent linearization method, it will be appropriate to use the equivalent stiffness K_B and the equivalent damping ratio h_B which is equivalent to the displacement of about 70% of the maximum response displacement.

(2) Since the basic mechanism of seismic force reduction by utilizing an isolation bearing results from creating a longer natural period through an isolator and absorbing the energy by a damper, the stiffness and the damping ratio of the isolation bearing shall be determined appropriately. However, since the relationship between the horizontal force and the horizontal displacement of an isolation bearing is generally non-linear, the calculation equations for expressing the stiffness and damping ratio of an isolation bearing corresponding to the effective design displacement by using the equivalent linearization method is provided.

Different values can be obtained for the effective design displacement u_{Be} of an isolation bearing for the verification of seismic performance under Level 1 and Level 2 Earthquake Ground Motion, respectively. Two different values can also be obtained for the equivalent stiffness K_B and the equivalent damping ratio h_B according to the effective design displacement u_{Be} of the isolation bearing.

Generally, the nonlinear characteristics of an isolation bearing can be expressed by a bilinear model illustrated in Figure C8.3.3-1. In this case, the equivalent stiffness and the equivalent damping ratio of the isolation bearing shall be obtained as follows:

$$K_B = \frac{Q_d}{u_{Be}} + K_2 \qquad \text{.........} \qquad \text{(C8.3.3-1)}$$

$$h_B = \frac{2Q_d \{u_{Be} + Q_d / (K_2 - K_1)\}}{\pi u_{Be}(Q_d + u_{Be}K_2)} \quad \text{(C8.3.3-2)}$$

where:

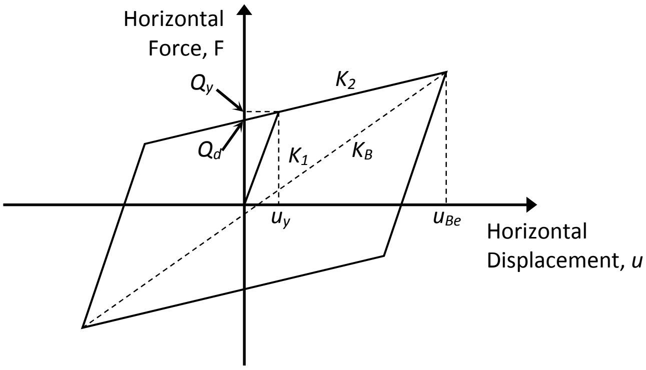
K_B : Equivalent stiffness of an isolation bearing, (kN/m).

h_B : Equivalent damping ratio of isolation bearing.

Q_d : Yield load of an isolation bearing as shown in Figure C8.3.3-1, (kN).

u_{Be} : Effective design displacement of an isolation bearing, (m).

K_1, K_2 : Initial and secondary stiffness of isolation bearing as shown in Figure C8.3.3-1, (kN/m)



K_B : Equivalent stiffness of isolation bearing (kN/m)
K_1, K_2 : Initial and secondary stiffness (kN/m)
Q_y : Yield horizontal force (kN)
Q_d : Yield load of isolation bearing (kN)
u_y : Yield horizontal displacement (m)
u_{Be} : Effective design displacement of isolation bearing (m)

Figure C8.3.3-1 Hysteretic Property of an Isolation Bearing

8.4 PERFORMANCE REQUIREMENT FOR ISOLATION BEARINGS

(1) The variation of the equivalent stiffness of an isolation bearing shall be within ± 10% of the value obtained from Article 8.3.3 of this Section. On the other hand, the equivalent damping ratio of an isolation bearing shall be greater than the value calculated from Article 8.3.3 of this Section.

(2) An isolation bearing shall be stable when subjected to repeated load corresponding to the design displacement u_B, which is calculated in the verification for Level 2 Earthquake Ground Motion, as specified in Article 8.3.3 of this Section.

(3) An isolation bearing shall possess positive tangential stiffness within the range of the design displacement u_B, calculated in the verification for Level 2 Earthquake Ground Motion, as specified in Article 8.3.3 of this Section.

(4) An isolation bearing shall generally prevent the occurrence of residual displacement affecting post-earthquake bridge functions.

(5) The equivalent stiffness and the equivalent damping ratio of an isolation bearing shall be stable for the environmental conditions including repeated live loads and temperature changes.

Commentary C8.4

(1) Although the indices to determine the dynamic characteristics of an isolation bearing include the equivalent stiffness, the equivalent damping ratio and the yield load, the effects of seismic isolation are basically dependent on the equivalent stiffness and the equivalent damping ratio of the isolation bearing. Accordingly, the change in the value of the equivalent stiffness of the isolation bearing corresponding to the effective design displacement u_{Be} of the isolation bearing specified in Article 8.3.3, which is used for verification for Level 2 Earthquake Ground Motion, shall be limited to within ± 10% of the design value. Besides, it is a rule that an isolation bearing with an equivalent damping ratio well above the design value shall be selected for use.

Here, the equivalent stiffness and the equivalent damping ratio of the isolation bearing refer to the average respective values calculated from 10 repeated alternating loads by using the effective design displacement u_{Be} of the isolation bearing specified in Article 8.3.3 for Level 2 Earthquake Ground Motion. Once the average values of the equivalent stiffness and the equivalent damping ratio of the isolation bearing meet the conditions specified above in relation to the design values, the variation in the range of the response acceleration or response displacement of the superstructure will not cause any particular problem in design

In addition, an isolation bearing that displays a big deviation in the values of the dynamic characteristics or which does not give a stable hysteretic curve must not be used.

(2) An isolation bearing may experience a rise in temperature and drop in energy absorption performance caused by the accumulation of absorbed energy, as well as a loss of energy absorption capacity caused by fracture. Therefore, it is stipulated that the energy absorption capacity of an isolation bearing shall be designed to be sufficiently larger than the energy input of the earthquake.

For Level 2 Earthquake Ground motion, the standard for the number of times of repeated alternating loads are approximately 50 times when taking into account Type I Earthquake (plate boundary type earthquake with large magnitude) and approximately 15 times when considering Type II Earthquake (inland direct strike type/fault movement earthquake). Some points still remain unclear about the frequency of repeated alternating loads in response to different types of seismic ground motions. However, it is generally known that the earthquake ground motions in a large-scale earthquake of the plate boundary type origin (Type I Earthquake) tend to have a high repeated frequency. On the other hand, for an earthquake of inland near-field type (Type II Earthquake) similar to the Hyogo-ken Nanbu Earthquake (Japan), the repeated frequency is likely to be low. Here, it is anticipated that the repeated frequency for each earthquake cycle is approximately 30 times for Type I Earthquake and between seven and eight times for Type II Earthquake. In order to ensure the safety of the isolation bearing, bigger values than these have been provided.

(3) Even if a major displacement which falls into the nonlinear zone has occurred in an isolation bearing, the displacement shall not be allowed to accumulate in one direction during an earthquake. For this reason, it is required that only an isolation bearing that always has a positive tangential stiffness within the range of the design displacement u_B of the isolation bearing shall be selected for use in principle.

(4) A seismically-isolated bridge shall be designed to prevent the occurrence of the residual displacement that may badly affect the driving conditions on the bridge after an earthquake. The recoverability of the isolation bearing to its original position which is used for the verification of seismic performance for Level 2 Earthquake Ground Motion can be expressed by the residual displacement u_{BR} occurring when the isolation bearing is naturally released from the design displacement u_B and is made to vibrate freely. The standard for the residual displacement u_{BR} of the isolation bearing can be set at a level lower than 10% of the design displacement u_B of the isolation bearing specified in Article 8.3.3 as used for the Level 2 Earthquake Ground Motion. However, when it is difficult to conduct an experiment for the actual isolation bearing due to its large size or other conditions, a scale model can be used for the proper study.

(5) An isolation bearing shall possess a stable equivalent stiffness and equivalent damping ratio in the temperature range specified for other limit states. At the same time, when the state of the bearing in which the bearing possess a static pre-displacement due to creep or shrinkage contraction is taken as the starting point, or even after the bearing is subjected to the repeated alternating loads of expansion and contraction of the girder caused by temperature changes and of vibration due to live loads, the equivalent stiffness and the equivalent damping ratio shall remain stable.

8.5 OTHER STRUCTURES FOR REDUCING EARTHQUAKE EFFECTS

When reduction of seismic response of a bridge is expected by using structures or devices other than seismic isolation, careful investigation shall be carried out on the dynamic characteristics and seismic behavior of the bridge while fully ensuring unseating prevention of the bridge.

Commentary 8.5

The seismic isolation design introduced in this Section is the design method to reduce the displacement of a girder by using a rubber bearing to provide flexible support for the superstructure in the horizontal direction and to create a long natural period. At the same time, energy absorbing devices such as a damper are also adopted in order to reduce the displacement of the girder. Design of the seismic isolation is based on the assumption that the rubber bearing and the energy absorbing devices are treated as a total seismic isolation bearing system.

Apart from the seismic isolation design specified in this Section, various new energy absorption devices, supporting structures and vibration control technologies such as active control and semi-active control are being developed. Since these cases are still uncommon at the present stage, concrete design details are not included here. Considering that this kind of technologies and trial cases of the new devices will be expected to increase in the coming years, only the general principle for designing the structures anticipated to reduce the effects of earthquake is provided in the current version.

When structures or devices for which the reduction of the effects of the earthquake forces are adopted, the following conditions shall be satisfied:

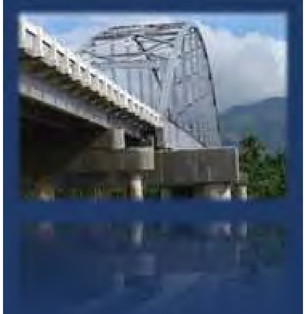
(1) It shall be simple in mechanism and can be used within the scale in which its mechanical properties are well understood.

(2) It shall be able to function in a stable condition under the Level 2 Earthquake Ground Motion.

(3) Verification of seismic performance using the dynamic analysis shall be carried out to evaluate the vibration characteristics of the whole bridge.





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