THE REPUBLIC OF THE PHILIPPINES
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS (DPWH)

THE PROJECT FOR STUDY ON IMPROVEMENT OF BRIDGES THROUGH DISASTER MITIGATING MEASURES FOR LARGE SCALE EARTHQUAKES IN THE REPUBLIC OF THE PHILIPPINES

FINAL REPORT

APPENDIX 1-A

PROPOSED DPWH BRIDGES SEISMIC DESIGN SPECIFICATIONS (DPWH-BSDS)

DECEMBER 2013

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

CTI ENGINEERING INTERNATIONAL CO., LTD
CHODAI CO., LTD.
NIPPON KOEI CO., LTD.

APPENDIX 1-A

PROPOSED DPWH BRIDGES SEISMIC DESIGN SPECIFICATIONS (DPWH-BSDS)

DPWH Guide Specifications

LRFD Bridge Seismic Design Specifications

1st Edition, 2013

Department of Public Works and Highways
Republic of the Philippines

The DPWH Guide Specifications for Seismic Bridge Design is prepared under the

Project for Study on Improvement of Bridges through Disaster Mitigating Measures for Large Scale Earthquakes in the Republic of the Philippines

December 2013

Japan International Cooperation Agency

Consultants:

CTI Engineering International Co., Ltd.
Consulting Engineers

CHODAI CO.,LTD.
CONSULTING ENGINEERS & PLANNERS

NIPPON KOEI
Challenging mind, Changing dynamics

FOREWORD

The Project "*Study on Improvement of Bridges through Disaster Mitigating Measures for Large-Scale Earthquakes*" is undertaken in response to the request of the Government of the Republic of the Philippines to the Government of Japan to improve the durability and safety bridges under large-scale earthquake in the Philippines. The Government of Japan thus decided to dispatch the Study Team, headed by Dr. Shingo Gose of CTI Engineering International Co., Ltd., under the Japan International Cooperation Agency (JICA) to collaborate with the Department of Public Works and Highways (DPWH) and prepare the bridge improvement plan that will be highly durable and safe against large-scale earthquakes.

Since the Philippines and Japan share a common geologic setting – with volcanoes, inland faults and ocean trenches, both countries are geographically prone to natural disasters, particularly large-scale earthquakes. And with the recent large-scale earthquake events experienced by both the Philippines and Japan, causing loss of lives and properties, it is important to safeguard the safety of important lifeline infrastructures such as bridges that links both the economic and social life of the people. The challenge now lies in building bridges more resilient to large-scale earthquakes.

One of the major tasks in this project is to improve the seismic design guidelines for bridges, which has been outdated by the occurrence of large-scale earthquakes and the technological advancement in the field of earthquake design and construction. Thus, the "*DPWH LRFD Bridge Seismic Design Specifications (BSDS)*" is developed under the project, incorporating Japan's experience in the design of bridges for large-scale earthquakes. However, design guides and specifications evolve with experience and technological advancement so that it is hoped that this BSDS will be the start of a more sustainable development of seismic design specifications in the Philippines.

Further, we hope that the BSDS will serve as a useful guide for the DPWH and the engineering professionals in the Philippines towards a safer and more durable design and construction of bridges.

Finally, we wish to express our sincere appreciation to the officials of the DPWH and the other agencies and associations for their close cooperation and contribution to this Study.

KAZUNORI MIURA
Director General
Economic Infrastructure Department
Japan International Cooperation Agency (JICA)

PREFACE

The entire Philippine archipelago is prone to various natural disasters such as earthquakes, typhoons, floods, landslides and others, of such scale and magnitude that the devastating effects are seen in the past and recent events bringing destruction to public and private infrastructures. Although mitigating these natural calamities and disasters are equally important to the safety and reliability of our transport infrastructures, the seismic performance of lifeline facilities such as bridges is considered a very important and urgent issue in design and construction. The impacts of the recent 2012 Magnitude 6.9 earthquake in Negros Oriental and the 2013 Magnitude 7.2 earthquake in Bohol to our roads and bridges justify the need to address and mitigate the effects of large-scale earthquakes.

However, the current seismic design policy and practice of bridges in the Philippines is based on the DPWH Department Order No. 75, which was issued on July 17, 1992, after recognizing the deficiencies in seismic design practice as seen from the failures of bridges and road structures during the 1990 North Luzon Earthquake with a Magnitude of 7.9. Since then, seismic design for bridges in the Philippines relies heavily on the AASHTO Standard Specifications combining the two-zone PGA (peak ground acceleration) map with the AASHTO response spectra. However, in view of the recent local and international earthquake events and the issues of advancement in engineering technology, the DPWH recognized the need to update its Design Guidelines, Criteria and Standards to be at par with the modern technologies that will mitigate earthquake forces demands based on the Philippine design conditions.

It is timely that the JICA Project on the "*Study on Improvement of Bridges through Disaster Mitigating Measures for Large-Scale Earthquakes*" is undertaken to enhance the performance of bridges under large-scale earthquake in the Philippines, including its safety and reliability. Under this project, the development of the "*DPWH LRFD Bridge Seismic Design Specifications*" to update the DPWH Guide Specifications is done considering the possible sources of ground motion – including faults and ocean trenches.

The *DPWH Bridge Seismic Design Specifications* include some provisions of the Japan Road Association (JRA), incorporating Japan's years of design and construction experience to mitigate large and destructive earthquakes. Some of the major notable changes in these Specifications include – the levels of bridge seismic performance in relation to its function and operational class, the development of the localized seismic spectral acceleration maps and the seismic design response spectra considering the effects of local ground conditions and the inclusion of ground liquefaction and lateral spreading effects in design.

We hope that, with this Specifications, our lifeline structures – bridges in particular, will be more resilient to large-scale earthquakes considering structural reliability and public safety.

Lastly, the Department wishes to extend our sincerest appreciation to the Government of Japan and the Japan International Cooperation Agency for all their support towards developing the *DPWH LRFD Bridge Seismic Design Specifications* and their contributions to improving the technical skills of our engineers in seismic design.

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INTRODUCTION

1.1 BACKGROUND

(1) The current design standards and procedures for all public infrastructure projects undertaken by the Department of Public Works and Highways (DPWH) is contained in a four-volume, 12-parts "*Design Guideline, Criteria and Standards for Public Works and Highways*" (DPWH Guidelines) published in 1982. The DPWH Guidelines incorporate the information, standards and methods for the design of highways, bridges, hydraulic structures (water supply, flood control and drainage), ports and harbors, and buildings (architectural, structural, sanitary, mechanical and electrical). The standards and guidelines are formulated to guide and set the minimum and acceptable limits in solving design problems and provide a more uniform design approach leading to a more efficient and economical design of various public infrastructure projects of the DPWH.

Part 4 – Bridge Design of the *DPWH Guidelines* contains the specifications and provisions for bridge design, including the minimum requirement for earthquake loading. However, since the guidelines are prepared in the early 1980s, the seismic design requirements and procedures are deficient and do not reflect the local conditions, realistic seismic forces and structural response under large-scale earthquakes. The devastating effects of the "*1990 North Luzon Earthquake*" noted such deficiencies in the seismic design of bridges in the Philippines which prompted the DPWH to issue the Department Order No. 75 (D.O. 75) requiring the seismic design of bridges to conform to the latest AASHTO Standard Specifications. In 2004, the DPWH attempted to incorporate the AASHTO seismic design procedures and guidelines for bridge retrofit with the DPWH Guidelines and issued a Draft Revision of *Part 4 – Bridge Design* of the DPWH Guidelines. However, this revision was not issued officially and remains a draft.

The current design practice of bridges under the DPWH (engineers and consultants) is to refer to the *AASHTO Standard Specifications for Highway Bridges (17th Edition, 2002)*, utilizing the load factor method, as the design specifications with minor revisions to suit local conditions. Design for earthquake forces is based on Division I-A (Seismic Design) of this Specification utilizing the AASHTO design seismic response spectra for Types I-IV AASHTO soil classification to model the seismic design forces. However, the peak ground acceleration (PGA) is based on the seismic zone map of the Philippines as specified in the *National Structural Code of the Philippines (NSCP) Vol. II (Bridges)*, 1997 with reprint in 2006. The design PGA coefficients are 0.2 for Palawan and Sulu and 0.4 for the rest of the country.

(2) Due to the urgent need to improve the seismic design guidelines in the wake of recent large earthquakes in the Philippines (including the 1990 North Luzon Earthquake, the 2012 Negros Oriental Earthquake and the 2013 Bohol Earthquake), the Japan International Cooperation Agency (JICA) undertook the project "*Study on Improvement of Bridges Through Disaster Mitigating Measures for Large Scale Earthquakes in the Republic of the Philippines*" which is aimed at enhancing bridge performance under large earthquakes, including its safety and durability. The development of the seismic design specifications for bridges to update the DPWH Guidelines for earthquake design is one of the main components of the project.

(3) The key features of the Bridge Seismic Design Specifications cover:

- establishment of the bridge seismic performance requirements and the bridge operational classification which is not previously defined in the DPWH Guidelines,

- localization of the Philippine design seismic response acceleration spectra through the development of design seismic peak ground acceleration and the response spectral acceleration coefficient maps, which is done for the first time in the Philippines,
- adoption of three (3) ground types for site classification and site effects to simplify ground conditions in seismic design,
- adoption of the *AASHTO LRFD Bridge Design Specifications* (2012 edition) as the base of these Specifications to update design procedures from load factor design to load and resistance factor design,
- adoption of applicable provisions of the *Japan Road Association (JRA) Specifications for Highway Bridges, Part V – Seismic Design* in soil liquefaction, foundation design, unseating prevention system and bridge seismic isolation, and
- reference to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2011) as an alternative Specification.

(4) The current efforts of the DPWH to address the issues of advancement in engineering technology include the project "*Enhancement of Management and Technical Processes for Engineering Design in the DPWH*". Notable key component of the project is the updating and revision of the existing DPWH Guidelines and the standard drawings for Surveys and Site Investigation (Vol. 1), Flood Control and Drainage Design (Vol. 3), Highways Design and Bridge Design (Vol. 4). The development of *Volume 4 – Bridge Design* covers bridge architecture, steel and concrete bridges, long span bridges, tunnels, bridge hydraulics, retrofitting of existing bridges and performance-based design, geo-hazard management, environmental safeguard, etc. However, although *Volume 4* will cover most aspects of bridge design, the *Bridge Seismic Design Specifications (BSDS)* developed under this JICA project will be used as the section for seismic design provisions.

(5) This Guide Specifications is thus prepared to cover the seismic design of new bridges and will guide the civil engineering professionals in the Philippines for the minimum requirements of seismic design of new bridges. Although some provisions of these Specifications may be applied to the retrofit of existing bridges, the policies and design requirements for seismic retrofit shall be dealt with separately as mandated by the DPWH.

Commentary C1.1

(1) The evolution of seismic design of bridges in the Philippines can be summarized as:

- The *DPWH Design Guidelines, Criteria and Standards for Public Works and Highways*, 1982 edition is a four-volume design guideline consisting of Part I – Survey and Investigation, Part II – Hydraulic, Part III – Highway Design and Part IV – Bridge Design. The DPWH Guidelines (with reference to 1977 AASHTO) was prepared by BOD when the DPWH was still a Ministry to establish an acceptable level of standards in the design, preparation of plans, specifications and related documents required of public infrastructure.

- Prior to the publication of the DPWH Guidelines, the DPWH refers to the earlier editions of the AASHO/AASHTO and the Ministry orders and memorandums to design highway bridges. As such, the seismic design of bridges in the Philippines is similar to the AASHTO design methodology with bridges constructed prior to 1960s having minimal or no seismic design considerations.

- In 1982, when the DPWH Guidelines was published, the seismic design provisions specifies that reference shall be made to the J.P. Hollings reports entitled "Earthquake Engineering for the Iligan-Butuan-Cagayan de Oro Road in the Island of Mindanao" and the "Earthquake Engineering for the Manila North Expressway Structures in Luzon, Philippines" to guide in determining the seismic forces and serves as a guide for earthquake design criteria. However, the calculated seismic design forces based on these reports shall not be less than the force produced by 10% (DL + ½LL) – where DL is the dead load and LL is the live load.

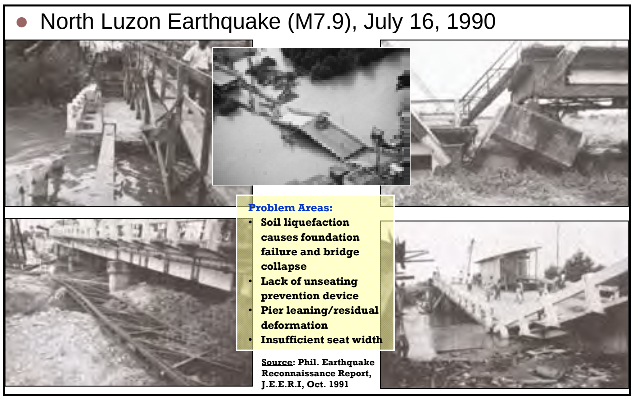


Figure C1.1-1 North Luzon Earthquake, 1990 (Philippines)

- In 1987, considering the development of seismic design codes in the USA in view of the damages to bridges caused by recent earthquake events, Association of Structural Engineers of the Philippines (ASEP) published the 1st Edition of the *National Structural Code of the Philippines (NSCP) Vol. 2 – Bridges* using the seismic design provisions of the 1983 AASHTO Standard Specifications.

- In 1990, the North Luzon Earthquake (Magnitude 7.9) caused major damages to public infrastructure in the Philippines including collapsed of several important highway bridges due to soil liquefaction effects and lateral spreading, lack of unseating prevention device and insufficient seat width, lack of column shear confinement and of capacity and foundation failure. Most bridges damaged by the earthquakes are those designed with minimal or no considerations for earthquake forces. Similar failure modes of bridges have been observed in the recent 2012 Negros Oriental Earthquake (Magnitude 6.9) and the 2013 Bohol Earthquake (Magnitude 7.2).

- Due the urgency of the need to establish proper seismic design considerations for bridges, the DPWH issued Department Order No. 75 in 1992 requiring the design of bridges to conform with the latest AASHTO seismic design provisions (1991 or later). This becomes the basis of the DPWH seismic design guidelines for new bridges until the present.

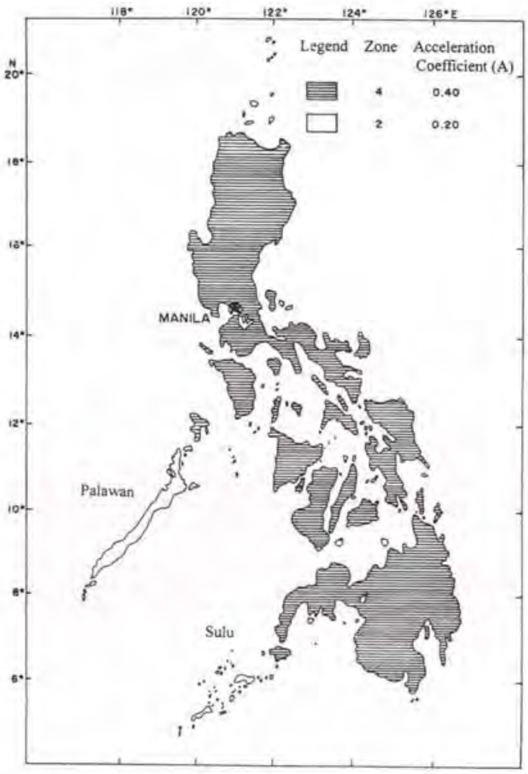


Figure C1.1-2 Philippine Seismic Zone Map

- In 1997, ASEP published the 2nd Edition of *NSCP Vol. 2 – Bridges*, utilizing the 1992 AASHTO Division I-A Seismic Design specifications as the seismic design section of the code. However, since there is no established data on ground accelerations in the Philippines, ASEP recommended a two-zone map for the entire Philippines to define the expected peak ground acceleration that will be used to determine the elastic seismic design forces. In the seismic zone map (Figure C1.1-2), the Philippines is under Zone 4 with acceleration coefficient (A) of 0.40, except for Palawan with A = 0.20.

- In 2004, DPWH internally issued the Draft "*Design Guidelines, Criteria and Standards for Public Works and Highways- Part IV Bridge Design*", owing to the need to update the seismic design specifications for DPWH bridge projects. This Guideline, however, refer to the ASEP seismic zone map of the Philippines for the ground acceleration coefficient A. Moreover, a section on “Guidelines for Seismic Retrofitting” was also added to guide the DPWH seismic retrofit projects. However, this Guideline was not officially issued and remains a draft.

- At present, the DPWH still refer to the ASEP seismic zone map for the ground response acceleration coefficient. Moreover, to determine the elastic design forces, DPWH uses the AASHTO normalized acceleration response spectra based on soil conditions in the USA. This becomes the drawback in the seismic design of bridges in the Philippines, indicating the need to generate a more realistic seismic zone map of ground acceleration and localized acceleration response spectra based on the actual soil conditions and site effects in the Philippines.

(2) Since the existing DPWH Guidelines published in 1982 have not been updated to address the advances in engineering technology, the design standards and techniques contained in the guidelines are outdated and in some cases do not represent the generally accepted design practices. With the objective of enhancing the engineering design process and upgrading the engineering design standards the DPWH is undertaking the project to update and improve the Design Guidelines, Criteria and Standards. The efforts of the DPWH to improve the seismic design guideline led to the development of the *DPWH Bridge Seismic Design Specifications* under the assistance from JICA.

(3) The commentary is intended to provide additional information to clarify and explain the technical basis for these Specifications and is not intended to be part of these Specifications. The following terms shall be interpreted as:

- The term "shall" denotes a requirement for compliance with these Specifications.
- The term "should" indicates a strong preference for a given criterion.
- The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.
- The term "recommended” is used to give guidance based on past experiences. Seismic design is a developing field of engineering that has not been uniformly applied to all bridge types; thus, the experiences gained to date on only a particular type are included as recommendations.

(4) These Specifications are intended for the design of new bridges. Although some of the provisions of these Specifications, including seismic hazard maps and design forces, may be used for bridge retrofit the guidelines and procedures shall be dealt with separately, as required by the DPWH.

1.2 PURPOSE OF SPECIFICATIONS

These Specifications are intended to:

1) Establish the design provisions for new bridges that will minimize susceptibility to damage from earthquakes and guarantee the required seismic performance level requirements of bridges,
2) Establish the minimum design earthquake forces to ensure safety and reliability of bridges, considering the local conditions inherent in the Philippines, and
3) Provide guidance to the DPWH engineers and the engineering professionals in the seismic design of new bridges that will set the minimum requirements for seismic design integrity and safety under a large earthquake.

Commentary C1.2

The need to localize some of the provisions for seismic design of the AASHTO Specifications is indicated due to the difference in seismicity and the geologic formation between the USA (based on AASHTO) and that of the Philippines. Specifically, this relates to the sources of seismic ground excitation (such as active faults and ocean trenches), soil formation and profiles, and earthquake recurrence and return periods inherent in the Philippines.

These Specifications are intended to guide the DPWH engineers and the design professionals for the minimum requirements in the design of new bridges under large earthquake as an extreme event. However, it does not limit the design engineers to employ new and advanced technologies in the design and construction of new bridges. Such technologies, which are not covered in these Specifications, shall be subject to the approval of the DPWH.

1.3 SCOPE OF SPECIFICATIONS

(1) The scope of these Specifications covers mainly seismic design of bridges under the "Extreme Event Limit State for Earthquake Loading (Extreme Event 1)" following the force-based R-factor (response modification factor) design concept and philosophy of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

Prior to the completion of the update and revision of the *DPWH Guide Specifications*, the design requirements for other limit states shall be that of the *AASHTO LRFD Bridge Design Specifications*.

(2) In these Specifications, the extreme event limit state shall be taken to ensure the structural survival of a bridge during a Level 2 Earthquake Ground Motion (commonly referred to as a large-scale or major earthquake).

(3) The analysis and design provisions employed by the *AASHTO Load and Resistance Factor Design (LRFD)* methodology shall be adopted unless stated explicitly in these Specifications. Bridges shall be designed for the specified limit states to achieve the objectives of constructability, safety and serviceability with due regard to issues of inspectability, economy, and aesthetics.

(4) The applicability of these Specifications for other provisions to the types of new bridges with regards to conventional structural form and construction method shall be as specified in the *AASHTO LRFD Specifications*. For non-conventional bridges and other types of construction (e.g. suspension bridges, cable stayed bridges, arch type bridges, and movable bridges), appropriate provisions of these Specifications may be adopted subject to prior approval by the DPWH.

(5) The provisions of these Specifications shall be taken as the minimum requirements for structural stability that is necessary to provide for public safety. When necessary, additional provisions may be specified by the DPWH to achieve higher performance criteria for repairable damage that may be attributed to critical or essential bridges. Where such additional requirements are specified, they shall be site or project specific and are tailored to a particular structure type. The DPWH may require, if necessary, the sophistication of design or the quality of materials and construction to be higher than the minimum requirements.

(6) The potential effects of unstable ground conditions (e.g. liquefaction, lateral spreading, landslides and slope movements, and fault displacements) on the on the structural stability and function of the bridge shall be considered.

(7) The design considerations to ensure that unseating or falling down of superstructures can be

prevented even if local failures of components or large displacement may occur due to unexpected structural behavior during an earthquake shall be as provided in these Specifications.

(8) Other provisions not contained in these Specifications shall be referred to the *AASHTO LRFD Bridge Design Specifications*. Further, reference is also made to the *Japan Road Association (JRA) Specifications for Highway Bridges Part V – Seismic Design* and *Part IV – Substructures* and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

Commentary C1.3

(1) The *AASHTO LRFD Specifications* require bridges to satisfy the corresponding limit states of serviceability, fatigue and fracture, strength and extreme events. However, in these Specifications, the coverage shall be the extreme event limit state during an earthquake. Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the life of the bridge. Since the DPWH is in the process of updating and revising the existing DPWH Guidelines and Specifications, the design for other limit states shall comply with the requirements of *AASHTO LRFD Specifications* until the updated DPWH Guidelines have been developed.

The development of these Specifications are based on the *AASHTO LRFD Bridge Design Specifications, 2012* (current version) utilizing the force-based approach to seismic design. The use of force-based approach (or commonly referred to as the *R-factor* method) has been widely used in the seismic design of bridges in the Philippines after the 1990 North Luzon Earthquake. Although *AASHTO Guide Specifications for LRFD Seismic Bridge Design* recently recommends the displacement-based approach as an alternate to seismic design of bridges, these Specifications will adhere to the force-based approach to account for column strength and ductility to which DPWH has more familiarity. The displacement-based alternative approach to seismic design is expected to be applied to DPWH bridges in the future.

The AASHTO Specifications recognize that since it is uneconomical to design a bridge to resist large earthquakes elastically, columns are assumed to deform inelastically where seismic forces exceed their design level. This is taken by dividing the elastically computed force effects by an appropriate response modification factor or commonly known as the force-reduction factor (R-factor). In this case, the structure, particularly columns, should have enough ductility to be able to deform inelastically to the deformations caused by large earthquakes without loss of post-yield strength. The force-reduction factors (R-factors) are then specified to determine the inelastic deformation demands on the bridge members when the design earthquake occurs.

The R-factors in these Specifications accounts for the strength and ductility requirements corresponding to the Seismic Performance Levels necessary for the performance of corresponding bridges classified under “Critical”, “Essential” and “Others” operational classifications (equivalent to the importance classification of the previous AASHTO Specification). The R-factors are then specified to determine the inelastic deformation demands on the bridge members when the design earthquake occurs.

(2) For the purposes of these provisions (in a similar manner with the AASHTO), conventional bridges have slab, beam, box girder, and truss for superstructures; have pier-type or pile-bent for substructures; and are founded on shallow-or piled footings or shafts. Substructures for conventional bridges are listed in Table 3.8.1-1. Non-conventional bridges include bridges with cable-assisted superstructures (cable-stayed or cable-suspended), bridges with truss towers or hollow piers for substructures, and arch bridges.

1.4 SEISMIC DESIGN PHILOSOPHY

(1) Bridges shall be designed to ensure the Seismic Performance required by the Operational Class and the Level of the design Earthquake Ground Motion in accordance with the provisions of Article 3.2 and Article 3.3.

(2) The design of bridges shall comply with the minimum concepts specified in the *DPWH Department Order No. 75 "DPWH Advisory for Seismic Design of Bridges"*, dated July 17, 1992 as follows:

1) Continuous bridges with monolithic multi-column bents have a high degree of redundancy and are the preferred type of bridge structure to resist shaking. Deck discontinuities such as expansion joints and hinges should be kept to an absolute minimum. Suspended spans, brackets, rockers, etc. are not recommended.

2) Where multi-span simple span bridges are justified, decks should be continuous.

3) Restrainers or unseating prevention device (horizontal linkage device between adjacent spans) are required at all joints and generous seat lengths at piers and abutments should be provided to prevent loss-of-span type failures.

4) Transverse reinforcements in the zones of yielding are essential to the successful performance of reinforced concrete columns during earthquakes. Transverse reinforcement serves to confine the main longitudinal reinforcement and the concrete within the core of the column, thus preventing buckling of the main reinforcement.

5) Plastic hinging should be forced to occur in ductile column regions of the pier rather than in the foundation unit. A scheme to protect the abutment piles from failure is often accomplished by designing the backwall to shear off when subjected to the design seismic lateral force that would otherwise fail the abutment piles. Plastic hinging occurring elsewhere shall be approved by the DPWH.

6) The stiffness of the bridge as a whole should be considered in the analysis. In irregular structures, as defined previously, it is particularly important to include the soil-structure interaction.

(3) Each component and connection shall satisfy Equation 1.4-1 for the extreme event limit state under earthquake effects.

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R = R_r \qquad (1.4\text{-}1)$$

In which:

- for loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad (1.4\text{-}2)$$

- for loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \qquad (1.4\text{-}3)$$

where:

γ_i : load factor; a statistically based multiplier applied to force effects

ϕ : resistance factor; a statistically based multiplier applied to nominal resistance specified in these Specifications and Sections 5, 6, 7, 8, 10, 11, and 12 of the *AASHTO LRFD Bridge Design Specifications*

η_i : load multiplier; a factor relating to ductility, redundancy, and operational classification

η_D : factor relating to ductility; η_D = 1.0 for extreme event limit state

η_R : factor relating to redundancy; η_R = 1.0 for extreme event limit state

η_I : factor relating to operational classification; η_I = 1.0 for all bridge class

Q_i : force effect

R_n : nominal resistance

R_r : factored resistance; ϕR_n

The resistance factors, ϕ, shall be taken as 1.0 except when stated in these Specifications or in the provisions of *AASHTO LRFD Bridge Design Specifications* corresponding to each seismic performance zones.

(4) The following shall be taken into account in the design of bridges:

1) topographic, geological, geotechnical/soil conditions, and other site conditions affecting seismic performance of the bridge,

2) selection of appropriate structural system with high seismic performance (see Commentary C1.4 (2)), and

3) assurance of seismic performance in the design of individual structural members of the bridge and the entire bridge system.

(5) The following two levels of Earthquake Ground Motions (EGM) shall be considered in these Specifications:

1) ***Level 1*** earthquake ground motion, considering seismic hazard from small to moderate earthquakes with high probability of occurrence during the bridge service life (100-year return), for ***seismic serviceability design objective*** to ensure normal bridge functions.

2) ***Level 2*** earthquake ground motion, considering a seismic hazard corresponding to an earthquake with return period event of 1,000 years (seven percent probability of exceedance in 75 years), for ***life safety performance objective*** under large earthquake.

Bridges shall resist Level 1 earthquake ground motion within the elastic range of the structural components without damage or with minor damage that can be repaired easily.

Life safety for the design event under Level 2 earthquake ground motion shall be taken to imply that the bridge has low probability of collapse but may suffer significant damage and that significant disruption to service is possible. Partial or complete replacement may be required. The DPWH may establish and authorize higher levels of performance, based on the operational objective.

(6) Earthquake loads shall be taken to be the horizontal force effects determined in accordance with the provisions of Article 4.1 to 4.3 and 5.2 to 5.3 of these Specifications on the basis of the elastic response coefficient, C_{sm}, specified in Article 3.6, and the equivalent weight of the superstructure, and adjusted by the response modification factor, R, specified in Article 3.8.

Commentary C1.4

(1) Bridges play a very important role as evacuation routes and emergency routes, allowing movement of facilities for first aid, medical services, firefighting and transport of urgent goods during a major disaster such as an earthquake. In this regard, it is essential to ensure the safety of the bridge in seismic design minimizing as much as possible the loss of bridge functions that might affect the lives of the community in general. Therefore, it is necessary to ensure the seismic performance required depending on the levels of earthquake ground motion and the bridge operational classification during seismic design.

The seismic design philosophy stipulates that the design earthquake forces and motions specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges that are designed and detailed in accordance with these provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking. For special bridges, the DPWH may choose to mandate higher levels of bridge performance.

(2) The selection of an appropriate bridge structural type and an earthquake resisting system (*ERS*) (as given in Appendix 1A) capable of meeting the requirements of seismic performance objectives is important in seismic design, considering the topographic, geologic and ground conditions, and other site conditions. Bridges shall then be designed with a structural system capable of fully resisting the earthquake forces utilizing the strength and ductility of the structural members. The following shall be considered in seismic design of bridges:

1) The following design concepts shall be adopted in addition to that required by the D.O. 75:

   a) Bearings that elastically distribute horizontal forces to the piers shall be preferred over fixed bearings to avoid excessive load on the pier that supports fixed bearings.

   b) For tall piers on mountainous areas, it is generally better to resist the seismic horizontal forces by the abutments rather than the piers if the ground conditions at the abutments are sufficiently sound.

   c) Accordingly, it is better to consider seismic performance of the entire bridge and to select proper bearing supports in view of bridge structural conditions and ground bearing properties. Horizontal seismic forces should be distributed, as much as possible, to all the piers in the entire bridge.

2) On reclaimed land or alluvial ground where ground deformation such as sliding of soft cohesive clayey layer, liquefaction of sandy layer and liquefaction-induced ground flow or lateral spreading may occur, a foundation with high horizontal stiffness should be designed. Further, a structural system such as multi-fixed-point type and rigid frame type, which has many contact points between superstructure and substructure should be selected.

3) For a multi-span, short period continuous bridge on stiff ground conditions, a seismically-isolated bridge is recommended to be adopted.

4) For a structure whose partial collapse may cause collapse of the entire system, the damage shall be limited to local failure.

5) For a strong earthquake motion, a proper structural system shall be designed by clarifying structural members with nonlinear behavior and those basically remaining in the elastic states. Additionally, a structure greatly affected by geometric nonlinearity or a structure having extensive eccentricity of dead loads, which tends to become unstable during a strong earthquake motion, shall not be adopted.

6) When ground conditions or structural conditions on a pier change remarkably, it is necessary to examine carefully which is more advantageous, either the case of two girder ends on the piers or that of a continuous girder.

(3) Equation 1.4-1 is the basis of *LRFD* methodology, with limit states specified to provide a buildable, serviceable bridge, capable of safely carrying the design loads for the design lifetime.

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

Ductility, redundancy and operational classification are considered in the load multiplier η. Whereas the first two directly relate to physical strength, the last (operational class) concerns the consequences of the bridge being out of service.

(5) The principles used for the development of the Specifications (adopting the AASHTO principles) are:

- Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage;
- Realistic seismic ground motion intensities and forces considering local site conditions should be used in the design procedures; and
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

(6) Earthquake loads are given by the product of the elastic seismic response coefficient C_{sm} and the equivalent weight of the superstructure. The equivalent weight is a function of the actual weight and bridge configuration and is automatically included in both the single-mode and multimode methods of analysis specified in Article 4.1 to 4.3. Design and detailing provisions for bridges to minimize their susceptibility to damage from earthquakes are contained in Sections 3, 4, 5, 6, 7, 10, and 11 of the *AASHTO LRFD Bridge Design Specifications 2012 (or later versions)*. However, the design and detailing of members and components shall also comply with the requirements of the updated DPWH Guidelines, once completed. A flow chart summarizing these provisions is presented in Article 1.6.

These Specifications are considered to be force-based wherein a bridge is designed to have adequate strength (capacity) to resist earthquake forces (demands). In recent years, there has been a trend to shift away from force-based procedures to those that are displacement-based, wherein a bridge is designed to have adequate displacement capacity to accommodate earthquake demands. Displacement-based procedures are believed to more reliably identify the limit states that cause damage leading to collapse, and in some cases produce more efficient designs against collapse. It is recommended that the displacement capacity of bridges designed in accordance with these Specifications, be checked using a displacement-based procedure, particularly those bridges in high seismic zones. The *AASHTO Guide Specifications for LRFD Seismic Design (2011)* offers the procedure for displacement-based approach.

1.5 LOADS AND LOAD AND RESISTANCE FACTORS FOR SEISMIC DESIGN

(1) The combination of factored extreme force effects for Extreme Event I load combination, as provided in Article 3.4 of the *AASHTO LRFD Bridge Design Specifications (2012)*, shall be adopted for seismic design in lieu of the on-going update and revision of the DPWH Guidelines (LRFD). Once completed, the load combination shall comply with the requirements of the DPWH Guidelines.

(2) The following loads, where applicable, shall be taken into account in the seismic design:

1) Permanent Loads

a. Dead load of structural components and nonstructural attachments (*DC*)

b. Dead load of wearing surface and utilities (*DW*)

c. Downdrag force (*DD*)

d. Miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction (*EL*)

e. Secondary forces from post-tensioning (*PS*)

f. Horizontal earth pressure load (*EH*)

g. Earth surcharge load (*ES*)

h. Vertical pressure from dead load of earth fill (*EV*)

i. Force effects due to creep (*CR*)

j. Force effect due to shrinkage (*SH*)

2) Transient Loads

a. Earthquake load (*EQ*)

b. Vehicular live load (*LL*)

c. Water load and stream pressure (*WA*)

d. Friction load (*FR*)

(3) The load combination for Extreme Event I limit state under earthquake loading shall be:

$$\text{Extreme Event I} = \gamma_p (DC+DW+DD+EL+PS+EH+EV+ES+CR+SH) + 1.0\,(WA) + 1.0\,(FR) + \gamma_{EQ}\,(LL) + 1.0(EQ) \quad \text{.........................} \quad (1.5\text{-}1)$$

where

γ_p : load factor for permanent loading

γ_{EQ} : load factor for live load applied simultaneously with seismic loads

All other applicable load notations are as stated in Item (2) above.

(4) The *AASHTO LRFD Bridge Design Specifications* provisions shall apply to both load and resistance factors, unless otherwise provided in these Specifications.

(5) The load factors for permanent loads, γ_p shall be that given in Table 3.4.1-2 and 3.4.1-3 of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

The load factor for live load effects, γ_{EQ} shall be 0.50.

Commentary C1.5

(1) These Specifications refers to Article 3.4 of the *AASHTO LRFD Bridge Design Specifications (version 2012)* that enumerates the loads and load combinations and load resistance factors for Extreme Event I relating to earthquake force effects. Since the DPWH is currently updating and revising its design guidelines for bridges and other structures, the AASHTO LRFD loads and load and resistance factors are used in these Specifications. However, when the DPWH Guidelines are completed, the load combinations and load and resistance factors specified in the DPWH Guidelines shall be used.

(2) Although the past editions of the AASHTO Standard Specifications does not consider live load in combination with earthquake forces, the previous DPWH Guidelines require 10% of ½ live load be considered in addition to the equivalent static earthquake force. However, in the latest editions of the *AASHTO LRFD Specifications*, the possibility of partial live load, i.e., γ_{EQ} = <1.0, with earthquakes is considered. Application of Turkstra's rule for combining uncorrelated loads indicates that γ_{EQ} = 0.50 is reasonable for a wide range of values of average daily truck traffic (*AADT*).

The use of γ_{EQ} = 0.50 is consistent with both the DPWH and AASHTO requirements.

1.6 SEISMIC DESIGN FLOWCHARTS

The flowcharts summarizing the seismic design provisions, are illustrated in Figures 1.6.1 and 1.6.2 below:

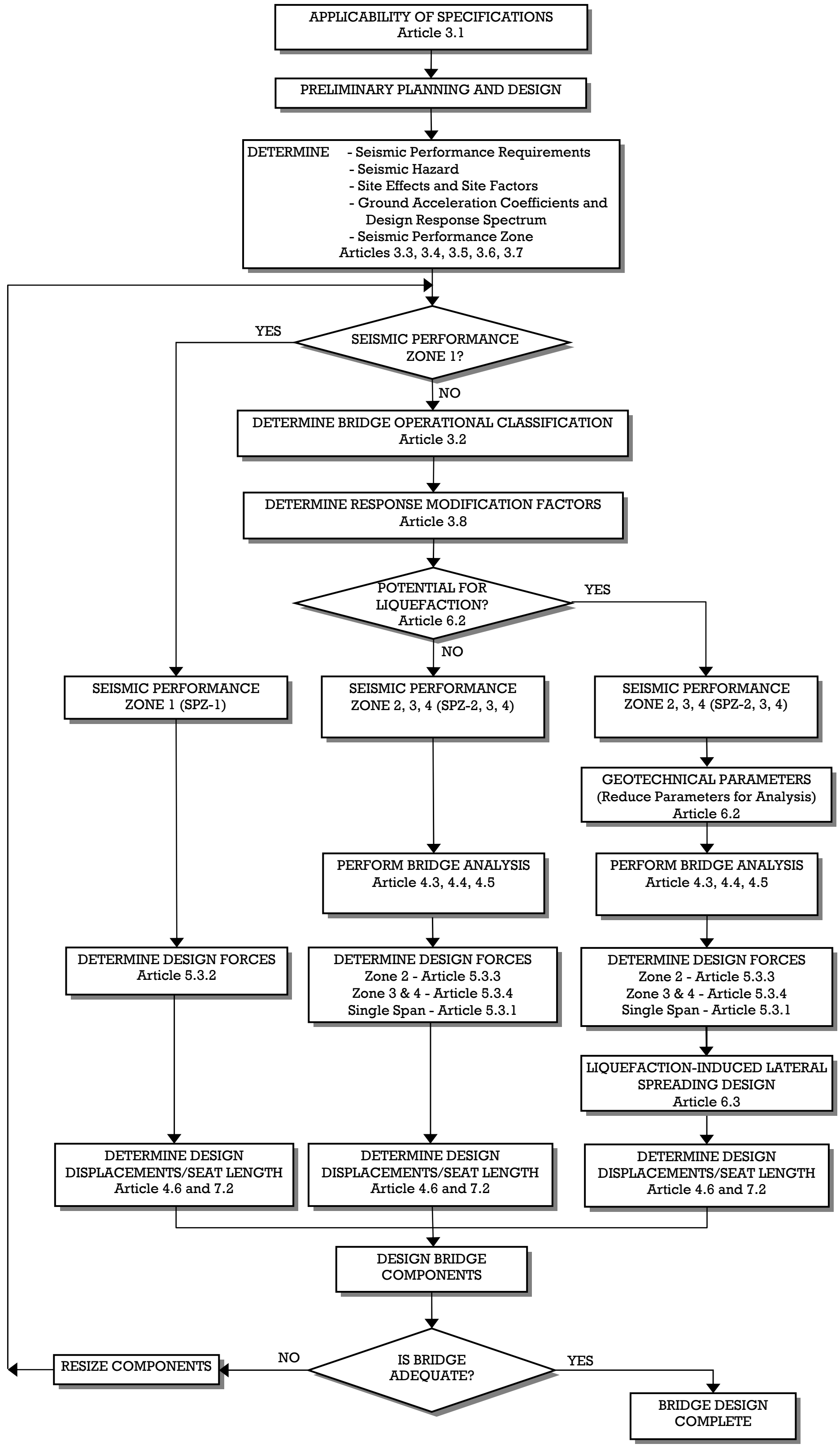


Figure 1.6-1 Seismic Design Procedure Flow Chart

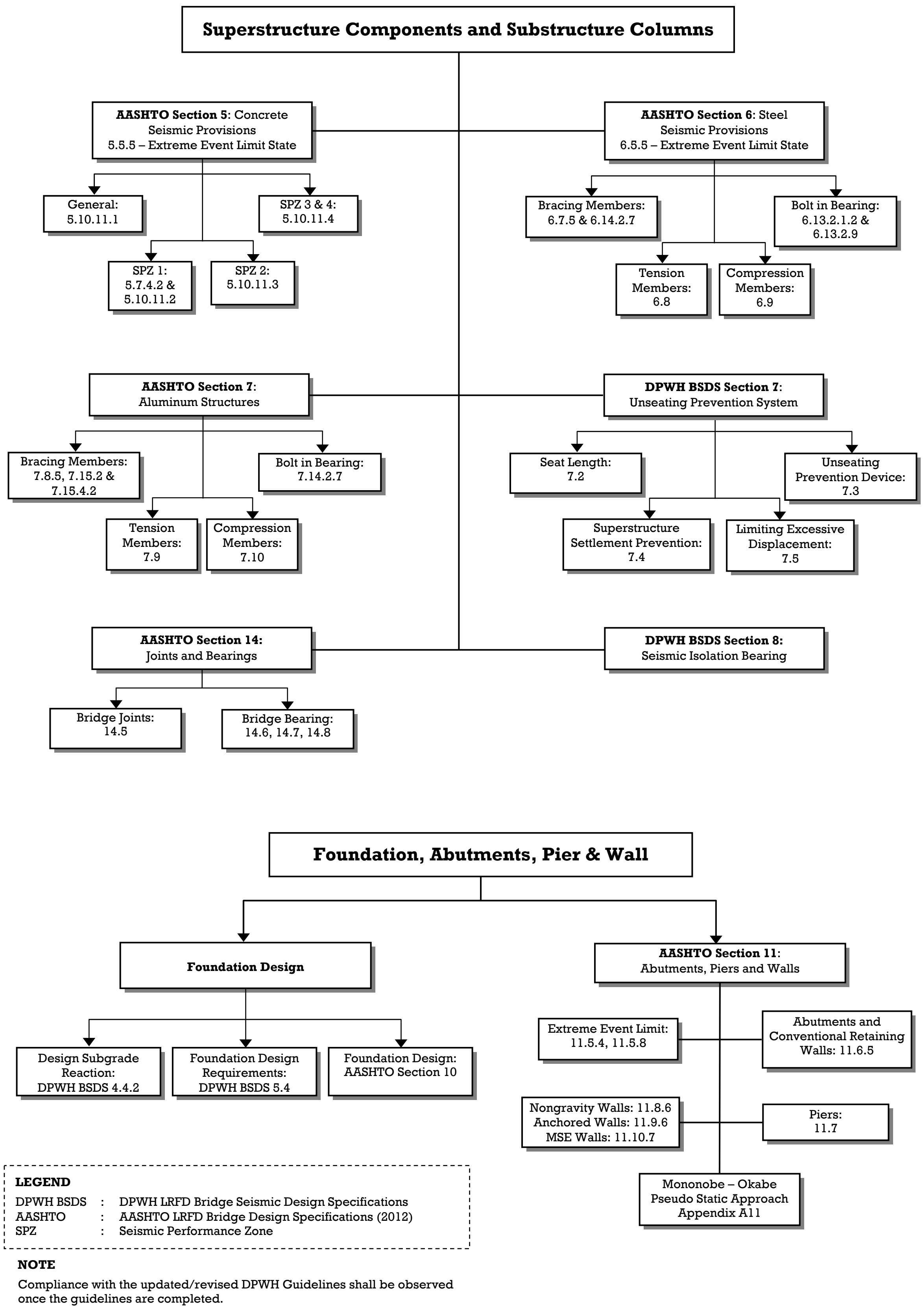


Figure 1.6-2 Seismic Detailing and Foundation Design Flow Chart

APPENDIX 1A:

EARTHQUAKE RESISTING SYSTEMS (ERS)

Appendix 1A *is taken from the* ***AASHTO Guide Specifications for LRFD Seismic Bridge Design (2011)*** *and is made as a reference guide for earthquake resisting systems (ERS) and earthquake resisting elements (ERE) in this Specification. Although the Guide Specifications for LRFD Seismic Bridge Design is based on the displacement-based design approach methodology, the ERS and ERE recommended in the AASHTO Specifications can also be applied to the force-based design approach as recommended in this Specifications.*

In the Global Seismic Design Strategies (GSDS) given in Item (3) below, ***Type 1*** *(Ductile Substructure with Essentially Elastic Superstructure) is widely used in the design of bridges in the Philippines under the DPWH. However, as practiced in the current design, the foundations are designed as elastic members capable of resisting the plastic forces generated in the piers or the elastic demand forces from the multimode analysis.* ***Type 2*** *(Essentially Elastic Substructure with a Ductile Superstructure) is not commonly used for seismic design in the Philippines whose application may be limited in these Specifications. However, the structural system behavior under seismic action shall be carefully considered when this type is being contemplated..* ***Type 3*** *(Elastic Superstructure and Substructure with a Fusing Mechanism between the Two) may be applied to the design of bridges where the use of this strategy will be advantageous to the structure.*

Type 2 and Type 3 GSDS will need DPWH prior approval before application to bridge design.

(1) *AASHTO Guide Specifications for LRFD Seismic Bridge Design (2011)* requires that for Seismic Design Categories (SDC) C or D (which is equivalent to the Seismic Performance Zone (SPZ) 3 and 4 in these Specifications), all bridges and their foundations shall have a clearly identifiable earthquake-resisting system (ERS) selected to achieve the life safety criteria. For SDC B (or SPZ 2 here), identification of an ERS should also be considered.

(2) The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

(3) Design should be based on the following three Global Seismic Design Strategies used in the Guide Specifications based on the expected behavior characteristics of the bridge system:

- **Type 1 –** <u>Ductile Substructure with Essentially Elastic Superstructure</u>: This category includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance. Also included are foundation that may limit inertial forces by in-ground hinging, such as pile bents and integral abutments on piles.

- **Type 2 -** <u>Essentially Elastic Substructure with a Ductile Superstructure</u>: This category applies only to steel superstructures, and ductility is achieved by ductile elements in the pier cross-frames.

- **Type 3 –** <u>Elastic Superstructure and Substructure with a Fusing Mechanism between the Two</u>: This category includes seismically isolated structures and structures in which supplemental energy-dissipation devices, such as dampers, are used to control inertia forces transferred between the superstructure and substructure.

(4) For the purposes of encouraging the use of appropriate systems and of ensuring due considerations of performance for the Owner, the ERS and earthquake-resisting elements (EREs) shall be categorized as follows:

- Permissible
- Permissible with Owner's approval, and
- Not recommended for new bridges

(5) These terms shall be taken to apply to both system and elements. For a system to be in permissible category, its primary EREs shall be in the permissible category. If any ERE is not permissible, then the entire system shall be considered not permissible.

(6) Permissible systems and elements depicted in Figures 1A-1a and 1A-1b shall have the following characteristics:

- All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. Piles subjected to lateral movement from lateral flow resulting from liquefaction are permitted to hinge below the ground line provided the Owner is informed and does not require any higher performance criteria for a specific objective. If all structural elements of a bridge are designed elastically, then no elastic deformation is anticipated and elastic elements are permissible, but minimum detailing is required according to the bridge seismic design category.
- Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g. cap beam and superstructure hinging).

(7) Permissible elements depicted in Figure 1A-2 that do not meet either criterion above may be used only with approval by the Owner.

(8) Examples of elements that do not fall in either of the two permissible categories depicted in Figure 1A-3 shall be considered not recommended. However, if adequate consideration is given to all potential modes of behavior and potential undesirable failure mechanisms are suppressed, then such systems may be used with the Owner's approval.

Commentary C1A

(1) Common examples from each of the three ERS and ERE categories are shown in Figures 1A-1a and 1A-1b, respectively. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase, or the type, size and location phase, or the design alternative phase of a project.

For SDC B (SPZ B), it is suggested that the ERS be identified. The displacement checks for SDC B (SPZ B) are predicated on the existence of a complete lateral load resisting system; thus, the Designer should ensure that an ERS is present and that no unintentional weak links exist. Additionally, identifying the ERS helps the Designer ensure that the model used to determine displacement demands is compatible with the drift limit calculation. For example, pile-bent connections that transmit moments significantly less than the piles can develop should not be considered as fixed connections.

(2) Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, may conflict with seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. For

example, resolution may lead to decreased skew angles at the expense of longer end spans. The resulting trade-off between performance and cost should be evaluated in the type, size and location phase, or design alternative phase, of a project, when design alternatives are viable from a practical viewpoint.

(3) There are a number of design approaches that can be used to achieve the performance objectives. These are discussed briefly below.

Type 1 – Ductile Substructure with Essentially Elastic Superstructure

Caltrans first introduced this design approach in 1973 following the 1971 San Fernando earthquake. It was further refined and applied nationally in the 1983 *AASHTO Guide Specification for Seismic Design of Highway Bridges*, which was adopted directly from the (ATC, 1981). These provisions were adopted by AASHTO in 1991 as their standard seismic provisions.

This approach is based on the expectation of significant inelastic deformation (damage) associated with ductility ≥4.

The other key premise of the provisions is that displacements resulting from the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the linear elastic response spectrum. As diagrammatically shown in Figure 1A-1c, this assumes that Δ^L_C (displacement capacity taken along the local principal axis corresponding to Δ^L_D of the ductile member) is approximately equal to Δ^L_D (displacement demand taken along the local principal axis of the ductile member). Work by Miranda and Bertero (1994) and by Chang and Mander (1994a and 1994b) indicates that this is a reasonable assumption, except for short-period structures for which it is nonconservative. A correction factor to be applied to elastic displacements to address this issue is given in Article 4.3.3 *AASHTO Guide Specifications for LRFD Seismic Bridge Design.*

Type 2 – Essentially Elastic Substructure with a Ductile Superstructure

This category applies only to steel superstructures. The ductility is achieved by constructing ductile elements as part of the cross-frames of a steel slab-on-girder bridge superstructure. The deformation capacity of the cross-frames located at each pier permits lateral displacement of the deck relative to the substructure below. This is an emerging technology and has not been widely used as a design strategy for new construction.

Type 3 – Elastic Superstructure and Substructure with a Fusing Mechanism between the Two

This category comprises seismically isolated structures and structures in which energy-dissipation devices are used across articulation joints to provide a mechanism to limit energy buildup and associated displacements during a large earthquake. The two subcategories are discussed further below.

Seismic Isolation. This design approach reduces the seismic forces a bridge needs to resist by introducing an isolation bearing with an energy-dissipation element at the bearing location. The isolation intentionally lengthens the period of a relatively stiff bridge, and this results in lower design forces, provided the design is in the decreasing portion of the acceleration response spectrum. This design alternative was first applied in the United States in 1984 and has been extensively reported on at technical conferences and seminars and in the technical literature. AASHTO adopted *Guide Specifications for Seismic Isolation Design of Highway Bridges* in 1991, and these have subsequently been revised. The current revisions are now referred to in Section 7 of these Guide Specifications. Elastic response of the substructure elements is possible with seismic isolation because the elastic forces resulting from seismic isolation are generally less than the reduced design forces required by conventional ductile design.

Energy Dissipation. This design approach adds energy-dissipation elements between the superstructure and the substructure and between the superstructure and abutment, with the intent of dissipating energy in these elements. This eliminates the need for energy dissipation in the plastic hinge zones of columns. This design approach differs from seismic isolation in that additionally flexibility is generally not part of the system and thus the fundamental period of vibration is not changed. If the equivalent viscous damping of the bridge is increased above five percent, then the displacement of the superstructure will be reduced. In general, the energy-dissipation design concept does not result in reduced design forces, but it will reduce the ductility demand on columns due to the reduction in superstructure displacement (ATC, 1993). This is an emerging technology and has not been widely used as a design strategy for new construction.

Abutments as an Additional Energy-Dissipation Mechanism

In the early phases of the development of these Guide Specifications, there was serious debate as to whether or not the abutments would be included and relied on in the ERS. Some states may require the design of a bridge in which the substructures are capable of resisting the entire lateral load without any contribution from the abutments. In this design approach, the abutments are included in a mechanism to provide an unquantifiable higher level of safety. Rather than mandate this design philosophy here, it was decided to permit two design alternatives. The first is where the ERS does not include the abutments and the substructures are capable of resisting all the lateral loads. In the second alternative, the abutments are an important part of the ERS and, in this case, a higher level of analysis is required.

If the abutment is included as part of the ERS, this design option requires a continuous superstructure to deliver longitudinal forces to the abutment. If these conditions are satisfied, the abutments can be designed as part of the ERS and become an additional source for dissipating the bridge's earthquake energy. In the longitudinal direction, the abutment may be designed to resist the forces elastically using the passive pressure of the backfill. In some cases, the longitudinal displacement of the deck will cause larger soil movements in the abutment backfill, exceeding the passive pressures there. This requires a more refined analysis to determine the amount of expected movement. In the transverse direction, the abutment is generally designed to resist the loads elastically. The design objective when abutments are relied on to resist either longitudinal or transverse loads is either to minimize column sizes or reduce the ductility demand on the columns, accepting that damage may occur in the abutment.

When the abutment is part of the ERS, the performance expectation is that inelastic deformation will occur in the columns as well as the abutments. If large ductility demands occur in the columns, then the columns may need to be replaced. If large movements of the superstructure occur, the abutment backwall may be damaged and there may be some settlement of the abutment backfill. Large movements of the superstructure can be reduced with use of energy dissipators and isolation bearing at the abutments and at the tops of the columns.

In general, the soil behind an abutment is capable of resisting substantial seismic forces that may be delivered through a continuous superstructure to the abutment. Furthermore, such soil may also substantially limit the overall movements that a bridge may experience. This is particular so in the longitudinal direction of a straight bridge with little or no skew and with a continuous deck. The controversy with this design concept is the scenario of what may happen if there is significant abutment damage early in the earthquake ground motion duration and if the columns rely on the abutment to resist some of the load. This would be a problem in a long-duration, high-magnitude (greater than magnitude 7) earthquake. Another consideration is if a gap develops between the abutment and the soil after the first cycle of loading, due to the inelastic behavior of the soil when passive pressures are developed.

Unless shock transmission units (STUs) are used, a bridge composed of multiple simply supported spans cannot effectively mobilize the abutments for resistance to longitudinal force. It is recommended that simply supported spans not rely on abutments for any seismic resistance.

Because structural redundancy is desirable (Buckle et al., 1987), good design practice dictates the use of the design alternative in which the intermediate substructures, between the abutments, are designed to resist all seismic loads, if possible. This ensures that in the event abutment resistance becomes ineffective, the bridge will still be able to resist the earthquake forces and displacements. In such a situation, the abutments provide an increased margin against collapse.

(4) The classification of ERS and EREs into permissible and not recommended categories is meant to trigger consideration of seismic performance that leads to the most desirable outcome, that is, seismic performance that ensures, whenever possible, post-earthquake serviceability. To achieve such objective, special care in detailing the primary energy-dissipating elements is necessary. Conventional reinforced concrete construction with ductile plastic hinge zones can continue to be used, but designers should be aware that such detailing, although providing desirable seismic performance, will leave the structure in a damaged state following a large earthquake. It may be difficult or impractical to repair such damage.

Under certain conditions, the use of EREs that require the Owner's approval will be necessary. In previous AASHTO seismic specifications, some of the EREs in the Owner's approval category were simply not permitted for use (e.g., in-ground hinging of piles and shafts and foundation rocking). These elements are now permitted in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, provided their deformation performance is assessed.

This approach of allowing their use with additional analytical effort was believed to the preferable to an outright ban on their use. Thus, it is not the objective of these Guide Specifications to discourage the use of systems that require Owner approval. Instead, such systems may be used, but additional design effort and consensus between the designer and Owner are required to implement such systems. Additionally, these Guide Specifications do not provide detailed guidance for designing all such systems, for example Case 2 in Figure 1A-2. If such systems are used, then case-specific criteria and design methodologies will need to be developed and agreed upon by the Designer and the Owner.

Bridges are seismically designed so that inelastic deformation (damage) intentionally occurs in columns so that the damage can be readily inspected and repaired after an earthquake. Capacity design procedures are used to prevent damage from occurring in foundations and beams of bents and in the connections of columns to foundations and columns to the superstructure. There are two exceptions to this design philosophy. For the pile bents and drilled shafts, some limited inelastic deformation is permitted below the ground level. The amount of permissible deformation is restricted to ensure that no long-term serviceability problems occur from the amount of cracking that is permitted in the piles. It is costly and difficult problem to achieve a higher performance level from piles.

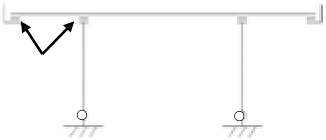
Longitudinal Response

1

- Plastic hinges in inspectable locations or elastic design of columns
- Abutment resistance not required as part of ERS
- Knock-off backwalls permissible

Longitudinal Response

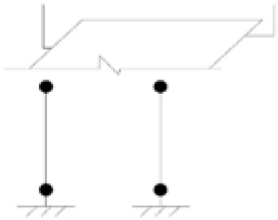
2



- Isolation bearings accommodate full displacement
- Abutment not required as part of ERS

Transverse Response

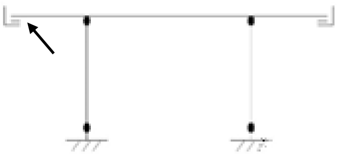
3



- Plastic hinges in inspectable locations
- Abutment not required in ERS, breakaway shear keys permissible

Transverse or Longitudinal Response

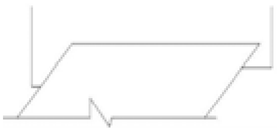
4



- Plastic hinges in inspectable locations
- Isolation bearing with or without energy dissipaters to limit overall displacements

Transverse or Longitudinal Responses

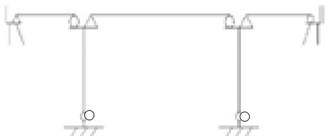
5



- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure shall be less than 0.70 of the value obtained using the procedure given in Article 5.2.3 (AASHTO Seismic Bridge Design)

Longitudinal Responses

6



- Multiple simple-supported spans with adequate support lengths
- Plastic hinges in inspectable locations

Figure 1A-1a – Permissible Earthquake-Resisting Systems (ERS)

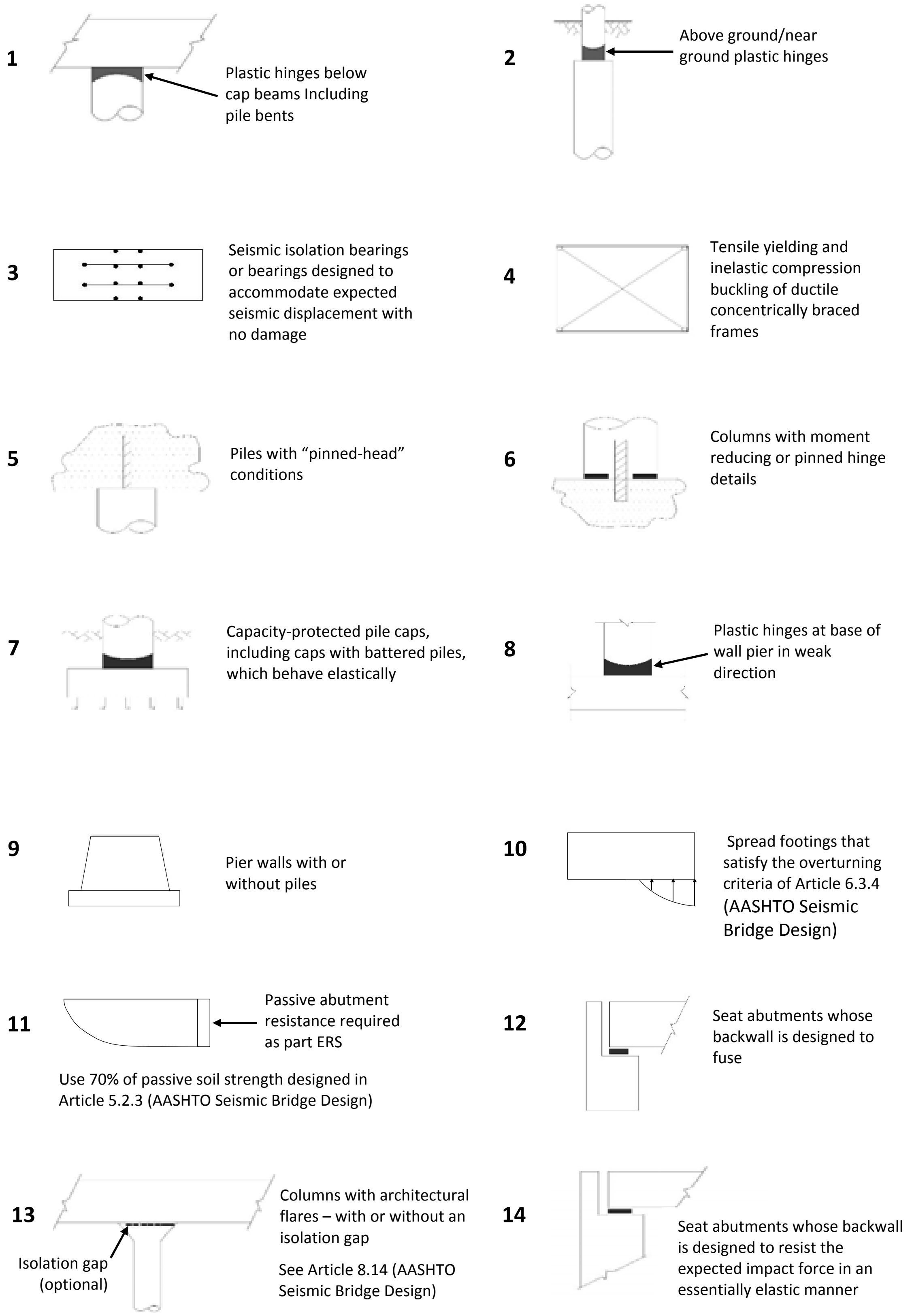


Figure 1A-1b – Permissible Earthquake-Resisting Elements (EREs)

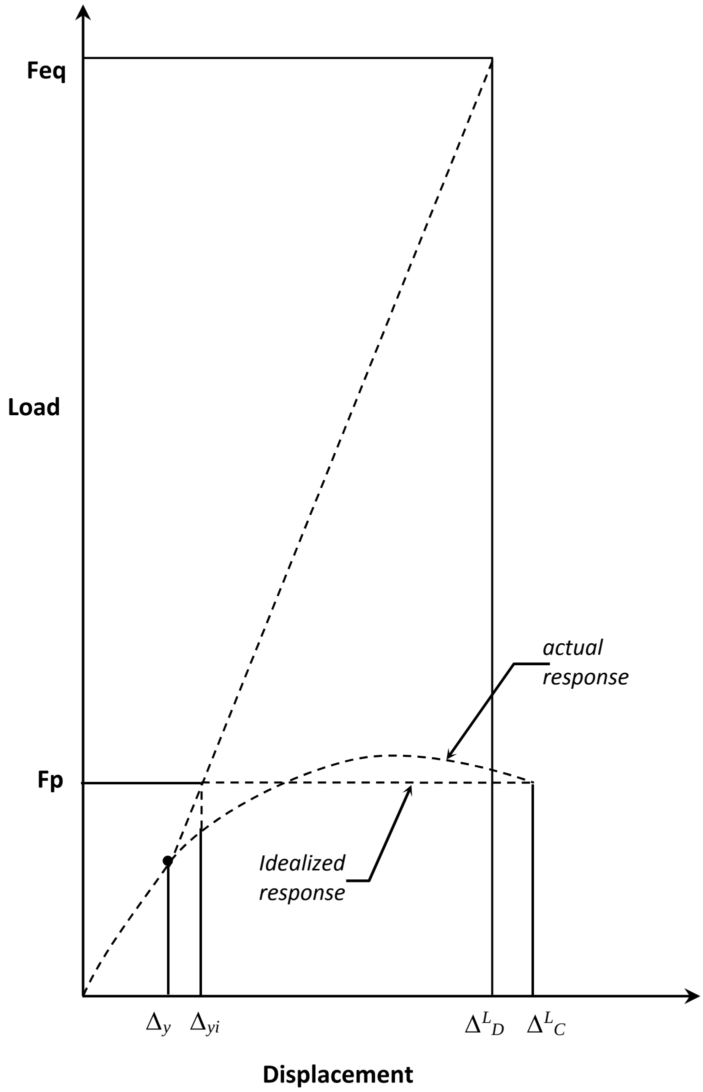


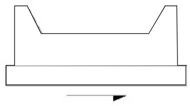
Figure 1A-1c – Design Using Strategy Type 1

1

Passive abutment resistance
Required as part of ERS
Passive Strength

Use 100% of strength designated in Article 5.2.3 (AASHTO Seismic Bridge Design)

2



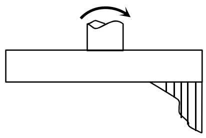
Sliding of spread footing, abutment allowed to limit force transferred

Limit movement to adjacent bent displacement capacity

3

Ductile end diaphragms in superstructure (Article 7.4.6 3, AASHTO Seismic Bridge Design)

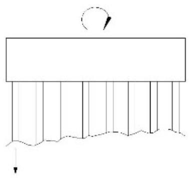
4



Foundation permitted to rock

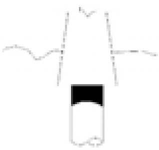
Use rocking criteria according to Appendix A of the Guide Specifications for LRFD Seismic Bridge Design (2011).

5



More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings

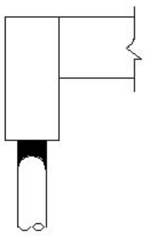
6



Wall piers on pile foundations that are not strong enough to force plastic hinging into the wall, and are not designed for the earthquake elastic forces

Ensure limited ductility response in piles according to Article 4.7.1 (AASHTO Seismic Bridge Design)

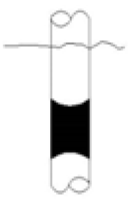
7



Plumb piles that are not capacity protected (e.g., integral abutment piles or pile-supported seat abutment that are not fused transversely)

Ensure limited ductility response in piles according to Article 4.7.1 (AASHTO Seismic Bridge Design)

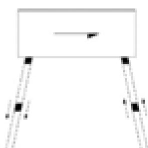
8



In-ground hinging in shafts or piles

Ensure limited ductility response in piles according to Article 4.7.1 (AASHTO Seismic Bridge Design)

9



Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms

Ensure limited ductility response in piles according to Article 4.7.1 (AASHTO Seismic Bridge Design)

Figure 1A-2 – Permissible Earthquake-Resisting that Require Owner's Approval

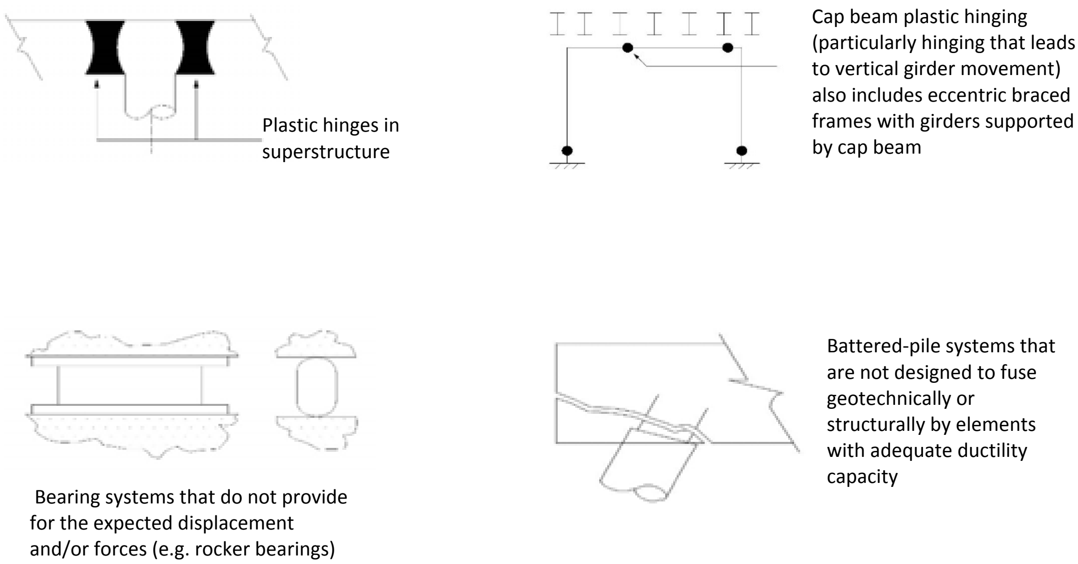


Figure 1A-3 – Earthquake-Resisting Elements that Are Not Recommended for New Bridges

SECTION 2:
DEFINITIONS AND NOTATIONS

2.1 DEFINITIONS

The definitions given below and those in the *AASHTO LRFD Bridge Design Specifications* apply to these Guide Specifications:

Active Earth Pressure – Lateral pressure resulting from the retention of earth by a structure or component that is tending to move away from the soil mass.

Capacity Design – A method of component design that allows the designer to prevent damage in certain components by making them strong enough to resist loads that are generated when adjacent components reach their overstrength capacity.

Compatibility – The geometrical equality of movement at the interface of joined components.

Component – A structural unit requiring separate design consideration; synonymous with member.

Critical Bridges – Bridges that must remain open to all traffic after the Level 2 design earthquake, i.e. 1,000 year return period event. Other bridges required by DPWH to be open to emergency vehicles and vehicles for security/defense purposes immediately after an earthquake larger than the Level 2 design earthquake (AASHTO recommends a 2,500-year return for larger earthquakes).

Damper – A device that transfers and reduces forces between superstructure elements and/or superstructure and substructure elements, while permitting thermal movements. The device provides damping by dissipating energy under seismic, braking or other dynamic loads.

Deformation – A change in structural geometry due to force effects, including axial displacement, shear displacement, and rotations.

Design – Proportioning and detailing of components and connections of a bridge to satisfy the requirements of these Specifications.

Design Earthquake – Refers to the two levels of design earthquake: Level 1 Earthquake Ground Motion with a 100-year return periods or a probability of exceedance corresponding to 53% in 75 years and Level 2 Earthquake Ground Motion with a 1,000 year return periods or a probability of exceedance corresponding to 7% in 75 years.

Earthquake Resisting Element (ERE) – The individual components, such as columns, connections bearings, joints, foundations and abutments, that together constitutes the earthquake-resisting system (ERS)

Earthquake Resisting System (ERS) – A system that provides a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Elastic – A structural material behavior in which the ratio of stress to strain is constant, the material returns to its original unloaded state upon load removal.

Element – A part of a component or member consisting of one material.

Equilibrium – A state where the sum of forces and moments about any point in space is 0.0.

Essential Bridges – Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes within a short period after the Level 2 design earthquake, i.e. 1,000 year return period event.

Extreme – A maximum or a minimum.

Finite Element Method – A method of analysis in which a structure is discretized into elements connected at nodes, the shape of the element displacement field is assumed, partial or complete compatibility is maintained among the element interfaces, and nodal displacements are determined by using energy variational principles or equilibrium methods.

Foundation – A supporting element that derives its resistance by transferring its load to the soil or rock supporting the bridge.

Inelastic – Any structural behavior in which the ratio of stress and strain is not constant, and part of the deformation remains after load removal.

Isolation System – A collection of all the elements that provide vertical stiffness, lateral flexibility, and damping to the system at the isolation interface. It includes the isolator units and the elastic restraint system, if one is used. The isolation system does not include the substructure and deck.

Isolator Unit – A horizontally flexible and vertically stiff bearing of the isolation system, which permits large lateral deformation under seismic load. The isolator unit may or may not provide energy dissipation.

Lateral Spreading – Lateral ground motion due to liquefaction.

Level 1 Earthquake Ground Motion – The design earthquake considering seismic hazard from small to moderate earthquakes with high probability of occurrence during the bridge service life (taken as 100-year return), for seismic serviceability design objective to ensure normal bridge functions.

Level 2 Earthquake Ground Motion – The design earthquake considering a seismic hazard corresponding to an earthquake with a return period event of 1,000 years (seven percent probability of exceedance in 75 years), for life safety performance objective under a large earthquake.

Life Safety Performance Level – The minimum acceptable level of seismic performance allowed by this Guide Specifications intended to protect human life following a rare earthquake. Life safety for the design event earthquake ground motion shall be taken to imply that the bridge has low probability of collapse but may suffer significant damage and that significant disruption to service is possible.

Linear Response – Structural behavior in which deflections are directly proportional to loads.

Liquefaction – The loss of shear strength in a saturated soil due to excess hydrostatic pressure. In saturated, cohesionless soils, such a strength loss can result from loads that are applied instantaneously or cyclically, particularly in loose fine to medium sands that are uniformly graded.

Liquefaction-Induced Lateral Flow – Lateral displacement of relatively flat slopes that occurs under the combination of gravity load and excess pore water pressure (without inertial loading from earthquake); often occurs after the cessation of earthquake loading.

Liquefaction-Induced lateral Spreading – Incremental displacement of a slope that occurs from the combined effects of pore water pressure buildup, inertial loads from the earthquake and gravity loads.

Load – The effect of acceleration, including that due to gravity, imposed deformation, or volumetric change.

Member – Same as *Component*.

Method of Analysis – A mathematical process by which structural deformations, force, and stresses are determined.

Minimum Seat Length – The minimum prescribed length of a bearing seat that is required to be provided in a new bridge designed according to this Guide Specifications.

Mode of Vibration – A shape of dynamic deformation associated with a frequency of vibration.

Model – A mathematical or physical idealization of a structure or component used for analysis.

Node – A point where finite elements or grid components meet; in conjunction with finite differences, a point where the governing differential equations are satisfied.

Nominal Load – An arbitrarily selected design load level.

Nominal Resistance – Resistance of a member, connection, or structure based on the expected yield strength (f_y) or other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Nonlinear Response – Structural behavior in which the deflections are not directly proportional to the loads due to stresses in the elastic range, or deflections causing significant changes in force effects, or by a combination thereof.

Operational Classification – The operational class required by DPWH for particular bridges specifies the minimum serviceability requirement of a bridge after the occurrence of a Level 2 design earthquake.

Overstrength Capacity – The maximum expected force or moment that can be developed in a yielding structural element assuming overstrength material properties and large strains with associated stresses.

Overstrength Moment Resistance – Denotes the moment factored resistance using the resistance factor ϕ of 1.3 for reinforced concrete columns and 1.25 for structural steel columns.

Passive Earth Pressure – Lateral pressure resulting from the earth's resistance to the lateral movement of a structure or component into the soil mass.

Seismic Performance Level – The level of performance in terms of post-earthquake service and damage that are expected to result from specified earthquake loadings if bridges are designed according to these Guide Specifications.

Permanent Loads – Loads and forces that are, or are assumed to be, either constant upon completion of bridge construction or varying only over a long time interval.

Plastic Hinge – The region of a structural component, usually a column or a pier in bridge structures, that undergoes flexural yielding and plastic rotation while still retaining sufficient flexural strength.

Plastic Hinge Zone – Those regions of structural components that are subject to potential formation of plastic hinge and thus shall be detailed accordingly.

Plasticity – A phenomenon of a structural member in which the member deforms beyond its elastic limit when subjected to seismic forces.

Repairability – This denotes capability to repair seismic damages. Bridges that are designed and detailed in accordance with these provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

Response Modification Factor (R-Factor) – Factors used to modify the element demands from an elastic analysis to account for ductile behavior and obtain design demands.

Restrainers – A system of high-strength cables or rods that transfers forces between superstructure elements and/or superstructure and substructure elements under seismic or other dynamic loads after an initial slack is taken up, while permitting thermal movements.

Safety – Bridge safety implies performance to avoid loss of life due to collapse or unseating of the superstructure during an earthquake.

Seismic Hazard Level – This is defined as a function of the magnitude of the ground surface shaking as expressed by F_vS_1.

Seismically Isolated Bridges – Bridge with seismic isolation bearings intended to make the natural period of the bridge longer and to increase the damping characteristics to decrease the inertia forces during an earthquake.

Seismic Performance – The performance of the bridge subjected to the effects of earthquake.

Seismic Performance Level 1 (SPL-1) – Performance level of a bridge to ensure its normal sound functions during an earthquake. This implies that the bridge shall be restrained safely from unseating, no emergency repair work is needed to recover the functions soon after the earthquake, and repair work which may take long time can be easily conducted. This indicates full access to all traffic immediately following an earthquake.

Seismic Performance Level 2 (SPL-2) – Performance level of a bridge to sustain limited damages during an earthquake and capable of recovery immediately for critical bridges and within a short period for essential bridges. This ensures not only safety in unseating prevention but also capability to recover the bridge functions soon after the event and repairability through comparatively easy, long-term repair work.

Seismic Performance Level 3 (SPL-3) – Performance level of a bridge to ensure safety against collapse during an earthquake. This implies that safety against collapse and unseating is ensured but does not cover the function necessary for serviceability and repairability for seismic design.

Seismic Performance Zones (SPZ) – Seismic zones that reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

Serviceability – This means that the bridge is capable of keeping its bridge functions such as fundamental transportation function, the role as evacuation routes and emergency routes for rescue, first aid, medical services, firefighting and transporting emergency goods to refugees.

Service Level – A measure of seismic performance based on the expected level of service that the bridge is capable of providing after the design earthquake.

Shock Transmission Unit (STU) – A device that provides a temporary rigid link between superstructure elements and/or superstructure and substructure elements under seismic, braking or other dynamic loads, while permitting thermal movements.

Site Class – One of the three ground types classification used to characterize the effect of the soil conditions at a site on ground motion.

Stiffness – Force effect resulting from a unit deformation.

Strain – Elongation per unit length.

Substructure – Structural parts of the bridge that support the horizontal span.

Superstructure – Structural parts of the bridge that provide the horizontal span.

Transient Loads – Loads and Forces that can vary over a short time interval relative to the lifetime of the structure.

Tributary Weight – The portion of the weight of the superstructure that would act on a pier participating in the ERS if the superstructure between participating piers consisted of simply supported spans. A portion of the weight of the pier itself may also be included in the tributary weight.

Uncracked Section – A section in which the concrete is assumed to be fully effective in tension and compression.

Unseating Prevention System – Structural system having a sufficient seat 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), in order to prevent the superstructure from unseating due to a large earthquake.

Unstable Ground – An extremely soft soil layer in seismic design, or a sandy layer affecting the bridge due to the liquefaction and lateral spreading.

Vibration Unit for Design – A structural system that can be regarded as a single vibration unit during an earthquake.

Yielding of foundation – Yielding of foundation 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.

2.2 NOTATIONS

The notations given below and those in the *AASHTO LRFD Bridge Design Specifications* apply to these Guide Specifications:

a	:	proportional coefficient
a_m	:	safety factor used for the calculation of the reinforced concrete columns
AADT	:	annual average daily truck traffic
AASHTO	:	American Association of State Highway and Transportation Officials (U.S.A.)
A_B	:	area of bottom surface of foundation, (m^2)
A_H	:	loading area of foundation perpendicular to the load direction, (m^2)
A_{HP}	:	effective projected vertical area of the ground corresponding to pile spring K_{HP}, (m^2)
A_i	:	block area at pile tip, (m^2)
A_P	:	area of bottom surface of each pile or pile tip, (m^2)
A_S	:	effective peak ground acceleration coefficient; effective area of bottom surface of foundation corresponding to spring K_S, (m^2)
A_{sc}	:	net cross-sectional area of soil cement column, (mm^2)
A_{sp}	:	net cross-sectional area of steel pipe, (mm^2)
A_{rr}	:	rotational spring constant due to foundation rotation, (kN-m/rad)
A_{rs}	:	spring constant in the rotational direction due to foundation horizontal translation, (kN-m/m)
A_{sr}	:	spring constant in the horizontal direction due to foundation rotation, (kN/rad)
A_{ss}	:	horizontal spring constant due to foundation horizontal translation, (kN/m)
A_V	:	loading area in the vertical direction, (m^2); effective area of bottom surface of foundation corresponding to spring K_V, (m^2)
b	:	total width of the superstructure, (m)
B	:	width of the foundation, (m)
B_e	:	effective loading width of foundation perpendicular to the load direction, (m)
B_H	:	equivalent loading width of foundation
B_V	:	equivalent loading width/diameter of foundation to be obtained by means of the following equation, (m)
c	:	cohesion of ground (kn/m2)
c_B	:	modification factor representing the dynamic properties of the inertia force and shall be taken as 0.70
c_F	:	design displacement coefficient of the unseating prevention structure (the standard value is taken as 0.75)
c_L	:	modification factor of the lateral movement force in a liquefying layer (can be taken as 0.3)
c_m	:	dynamic modification factor; dynamic modification factor used for the calculation of design displacement of an isolation bearing, and can be taken as 1.20
c_{NL}	:	modification factor of the lateral movement force in a non-liquefying layer
CQC	:	complete quadratic combination method
CR	:	force effects due to creep
c_s	:	modification factor on distance from the water front
C_s	:	coefficient of sliding between pile shaft and surrounding soil, (kN/m^3)
C_{sm}	:	elastic seismic response coefficient

c_V	:	modification factor based on the degree of ground strain
c_w	:	modification factor on earthquake ground motion
c_1, c_2	:	modification factors of N value on fine content
D	:	distance from fault, (m); loading width of foundation perpendicular to the load direction, (m); pile diameter, (m)
DC	:	dead load of structural components and nonstructural attachments
DD	:	downdrag force
D_f	:	effective embedded depth from design ground surface to the bottom of foundation, (m)
DPWH	:	Department of Public Works and Highways (Philippines)
DW	:	dead load of wearing surface and utilities
D_{50}	:	mean grain diameter (mm)
E_D	:	dynamic modulus of deformation of the ground, (kN/m^2)
EGM	:	earthquake ground motion
EH	:	horizontal earth pressure load
EI	:	flexural stiffness of foundation (kN•m^2)
EI_F	:	footing flexural rigidity
EI_{P}	:	substructure/pier flexural rigidity
EI_U	:	superstructure flexural rigidity
EL	:	miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
E_O	:	modulus of deformation of a soil layer at the point in issue (kN/m^2)
E_p	:	Young's modulus of pile, (kN/mm^2)
EQ	:	earthquake load
ES	:	earth surcharge load
E_{sc}	:	modulus of deformation of soil cement, (kN/mm^2)
E_{sp}	:	Young's modulus of steel pipe, (kN/mm^2)
EV	:	vertical pressure from dead load of earth fill
FC	:	fine content (%) (percentage by mass of fine soil passing through the 75μm mesh)
F_a	:	site coefficient for 0.20-sec period spectral acceleration
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)
f_i	:	maximum shaft resistance of soil layer considering pile shaft resistance, (kN/m^2)
F_L	:	liquefaction resistance factor
F_{pga}	:	site coefficient for the peak ground acceleration coefficient
FR	:	friction load
$F(u)$	:	horizontal force necessary to produce the horizontal displacement u of an isolation bearing, (kN)
F_v	:	site coefficient for 1.0-sec period spectral acceleration
g	:	acceleration of gravity (= 9.8 m/sec^2)
G_D	:	dynamic shear deformation modulus of deformation of the ground, (kN/m^2)
h	:	axial length of the pile above design ground surface, (m)
H	:	soil layer thickness
H_A	:	alluvial layer thickness

H_B	:	design horizontal seismic force of bearing support
h_B	:	equivalent damping ratio of an isolation bearing
H_D	:	diluvial layer thickness
H_F	:	design seismic force of the unseating prevention device, (kN)
H_i	:	thickness of the *i-th* soil layer, (m)
H_L	:	thickness of liquefying layer, (m)
H_{NL}	:	thickness of non-liquefying layer, (m)
h_s	:	vertical distance from the bearing seat surface to the center of gravity of the superstructure, (m)
H_S	:	design seismic force used in the design of the structure or device limiting excessive displacement, (kN)
H_0	:	lateral loads acting at the bottom of pile cap, (kN)
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)
h_w	:	depth of the ground water level, (m)
i	:	number of the *i-th* soil layer from the ground surface when the ground is classified into *n* layers from the ground surface to the surface of the base ground surface for seismic design
I_B	:	second moment of the bottom surface section of foundation, (m^4)
JRA	:	Japan Road Association
k	:	coefficient of subgrade reaction, (kN/m^3)
K	:	coefficient representing a proportional relationship
k_B	:	spring constant of elastic bearing support
K_B	:	spring constant of elastic bearing support; equivalent stiffness of an isolation bearing used for the calculation of u_B, (kN/m)
k_{Bi}	:	spring constant of the bearing support for the i-th design vibration unit, (kN/m)
k_h	:	design horizontal seismic coefficient for Level 1 Earthquake Ground Motion
k_H	:	coefficient of horizontal subgrade reaction, (kN/m^3); coefficient of subgrade reaction in the horizontal direction at the pile section corresponding to area A_{HP}, (kN/m^3)
k_{hgL}	:	horizontal seismic coefficient at the ground surface
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)
K_{HP}	:	horizontal spring constant of pile section corresponding to area A_{HP}, (kN/m)
K_p	:	passive earth pressure coefficient (in normal condition)
k_S	:	coefficient of horizontal shear subgrade reaction, (kN/m^3)
K_S	:	horizontal spring constant of pile, (kN/m)
k_v	:	coefficient of vertical subgrade reaction, (kN/m^3); coefficient of subgrade reaction in the vertical direction at the bottom of pile, (kN/m^3); design vertical seismic coefficient
K_V	:	axial spring constant of pile, (kN/m); total axial spring constant of the pile group, (kN/m)
K_{VP}	:	axial spring constant of a single pile in the direction of the pile axis, (kN/m)
k_{v0}	:	coefficient of vertical subgrade reaction equivalent to the value of plate bearing test using a rigid disc of diameter 0.3 m, and it 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_1 : initial stiffness of isolation bearing, (kN/m)

K_2 : secondary stiffness of isolation bearing, (kN/m)

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)

l : length of the effective span, (m)

L : total length of the bridge, (m); distance between adjacent pile centers, (m); length of pile (m); seismic shear stress ratio; distance between two substructures for determining the seat length (m); length of continuous superstructure, (m)

L_e : effective embedment depth of a foundation, (m)

L_i : thickness of soil layer considering shaft resistance, (m); thickness of each layer from the footing to the pile tip, (m)

LL : vehicular live load

LRFD : load and resistance factor design

L_A : allowance for the 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_S : gap distance of the structure limiting excessive displacement, (mm)

L_{Sd} : design gap distance of the structure limiting excessive displacement, (mm)

L_θ : length of a continuous superstructure, (m)

MM : multimode elastic method for multi-span bridge structures

M_n : nominal flexural strength of column or pier calculated at the axial load on the column or pier, (N-mm)

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

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

n : total number of piles; number of rubber layers

N : N-values from the Standard Penetration Test (SPT)

N_a : modified N value taking into account the effects of grain size

N_c, N_q, N_γ : coefficients of bearing capacity

N_i : average *N-value* of the *i-th* soil layer obtained from SPT

N_1 : equivalent N value corresponding to effective overburden pressure of 100 kN/m^2

OC : operational classification

p : subgrade reaction per unit area, (kN/m^2)

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

p_o : a uniform load arbitrarily set equal to 1.0 (N/m)

PGA : peak horizontal ground acceleration coefficient on AASHTO Class B rock

PHIVOLCS : Philippine Institute of Volcanology and Seismology

P_{Hi} : radial force of the i-th pile, (kN)

P_{LG} : the horizontal strength of the substructure based on the yield bending moment or the shear strength capacity of the substructure, (kN)

P_n	:	nominal axial pull-out resistance of pile, (kN)
P_{Ni}	:	axial force of the i-th pile, (kN)
P_R	:	factored axial pull-out resistance of pile, (kN)
PS	:	secondary forces from post-tensioning
$PSHA$	:	probabilistic seismic hazard analysis
P_u	:	axial load on column or pier, (N); horizontal force equivalent to the lateral strength of a column or plastic shear forces when considering plastic behavior of the pier or column; or horizontal force equivalent to maximum response displacement of a foundation when considering plastic behavior of the foundation, (kN)
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)
q_d	:	nominal end bearing resistance intensity per unit area, (kN/m^2)
q_d'	:	nominal bearing capacity intensity of soil at the base of the foundation, (kN/m^2);
Q_d	:	yield load of an isolation bearing, (kN)
Q_f	:	nominal skin friction of pile group, (kN)
Q_i	:	force effect
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)
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_p	:	nominal end bearing capacity as pile group, (kN)
Q_R	:	factored axial bearing capacity of pile group (capacity at pile head), (kN)
q_u	:	unconfined compressive strength, (kN/m2)
Q_y	:	horizontal yield force at the same time yielding, (kN)
R	:	dynamic shear strength ratio
r_d	:	reduction factor of seismic shear stress ratio, in terms of depth
R_d	:	dead load reaction (in case the structure connects two adjacent girders, the larger reaction shall be taken), (kN)
R_D	:	reaction force generated in the bearing support by the dead load of the superstructure, (kN)
R-factor	:	response modification factor;
R_{HEQ}	:	vertical reaction force generated in the bearing support system when the action of the horizontal force is applied on the bearing support in the transverse direction to the bridge axis, (kN)
R_L	:	cyclic triaxial shear stress ratio; downward seismic force used for seismic design of bearing support system, (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_n	:	nominal resistance; nominal resistance of pile, (kN)
R_r	:	factored resistance; ϕR_n
R_R	:	factored resistance of pile, (kN)
R_U	:	upward seismic force used for seismic design of bearing support system, (kN);
R_{VEQ}	:	vertical seismic force generated by the design vertical seismic coefficient k_v, (kN)
S_a	:	design response spectral acceleration coefficient
S_{D1}	:	design earthquake response spectral acceleration coefficient at 1.0-sec period

S_{DS}	:	design earthquake response spectral acceleration coefficient at 0.20-sec period
S_E	:	seat length of the girder at the support, measured from the end of girder to the edge of the top of the substructure, or the girder length on the hinge/bearing joint, (m)
$S_{E\theta}$	:	seat length for a skew bridge, (m)
$S_{E\phi}$	:	seat length for a curved bridge, (m)
S_F	:	maximum design allowance length of the unseating prevention device, (m).
SH	:	force effect due to shrinkage
SM	:	single-mode elastic method for multi-span bridge structures
SPL	:	seismic performance level
SPT	:	standard penetration test
SRSS	:	square root of the sum of the squares method
S_S	:	0.20-sec period spectral acceleration coefficient on AASHTO Class B rock
SPZ	:	seismic performance zone
STU	:	shock transmission unit
S_1	:	1.0-sec period spectral acceleration coefficient on AASHTO Class B rock
T	:	period of fundamental mode of vibration, (s)
t_{ei}	:	thickness of the *i*-th layer of the rubber, (mm)
T_F	:	bridge fundamental period
T_G	:	characteristic value of ground, (s)
TH	:	time history method of analysis for multi-span bridge structures
T_m	:	period of vibration of *m*th mode, (s)
T_0	:	reference period used to define spectral shape = $0.2T_s$ (s)
T_S	:	corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (s)
U	:	perimeter of pile, (m)
u_B	:	design displacement of an isolation bearing, (m); horizontal displacement generated in the bearing support by the design horizontal seismic force, (mm)
u_{Be}	:	effective design displacement of an isolation bearing, (m)
u_{Bi}	:	relative displacement at the bearing support system representing the i-th design vibration unit, (m)
u_{Fi}	:	horizontal displacement at the height of superstructure inertial force due to the displacement of the pier foundation representing the i-th design vibration unit included in the effects of liquefaction and lateral spreading of the ground, (m)
u_G	:	relative displacement of the ground caused by seismic ground strain, (m)
UL	:	uniform load elastic method for multi-span bridge structures
u_{Pi}	:	response displacement of the column representing the i-th design vibration unit, (m)
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)
u_{Ri}	:	displacement of the i-th design vibration unit, (m)
USGS	:	United States Geological Survey
u_y	:	horizontal yield displacement, (m)
v_D	:	dynamic Poisson's ratio of the ground
V_0	:	vertical loads acting at the bottom of pile cap, (kN)

V_{si} : average shear elastic wave velocity of the i-th soil layer, (m/s)

$\tilde{v}_s$: average shear wave velocity for the upper 30.5m of the soil profile, (m/s)

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

$v_s(x)$: deformation corresponding to p_o, (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)

W : effective weight of pile and soil inside pile, (kN); effective weight of soil replaced by the hypothetical caisson foundation, (kN); elastic energy of an isolation bearing, (kN·m)

WA : water load and stream pressure

W_F : weight of footing or pile, (kN)

W_s : effective weight of soil replaced by pile, (kN)

W_u : weight of superstructure, whose horizontal force is borne by the isolation bearing, (kN)

x : depth from the ground surface, (m)

x_i : x-coordinate of the i-th pile head, (m); horizontal distance from the gravity center of the superstructure to the i-th bearing support, (m)

x_o : distance from the balance point of R_{HEQi} to the center of gravity, (m)

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

α : coefficient to be used for estimating a coefficient of subgrade reaction; shape factor at bottom of foundation; rotational angle of the footing at the origin O, (rad); safety factor used for calculation of the allowable ductility ratio of the reinforced concrete columns

α_E : marginal unseating rotation angle, (degree); α_E can generally be taken as 5 degrees

β : characteristic value of pile/foundation, (m^{-1}); shape factor at bottom of foundation

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

γ : modification coefficient depending on nominal bearing resistance estimation method; shear strain generated in a rubber bearing or a seismic isolation bearing

γ_F : load factor for ground acceleration and shall be taken as 0.50

γ_i : load factor; a statistically based multiplier applied to force effects

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)

γ_t : unit weight of the ground, (kN/m^3)

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)

γ_1 : unit weight of soil layer below the foundation bottom, (kN/m^3)

γ_2 : unit weight of soil layer above the foundation bottom, (kN/m^3)

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)

ΔW : total energy absorbed by an isolation bearing (equal to the area inside the hysteretic curve of horizontal displacement-force relationship), (kN·m)

δ	:	displacement, (m)
δ_E	:	displacement of the superstructure toward the outside direction of the curve, (m)
δ_{Fi}	:	response horizontal displacement of the pier foundation representing the i-th design vibration unit, (m)
δ_x	:	lateral displacement from the origin O, (m)
δ_{xi}'	:	Radial displacement at the i-th pile head, (m)
δ_y	:	vertical displacement from the origin O, (m); yield displacement of the reinforced concrete columns
δ_u	:	ultimate displacement of the reinforced concrete columns
δ_{yi}	:	yielding displacement of the column representing the i-th design vibration unit, (m)
δ_{yi}'	:	Axial displacement at the i-th pile head, (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
η_D	:	factor relating to ductility; η_D = 1.0 for extreme event limit state
η_i	:	load multiplier; a factor relating to ductility, redundancy, and operational classification
η_I	:	factor relating to operational classification; η_I = 1.0 for critical and essential bridges and η_I = 1.0 for other bridge class
η_R	:	factor relating to redundancy; η_R = 1.0 for extreme event limit state
θ	:	skew angle, (degree); intersection angle, (degree)
θ_i	:	angle from the pile vertical axis of the i-th pile, (degree)
θ_{Fi}	:	response rotation angle of the pier foundation representing the i-th design vibration unit, (rad)
μ	:	modification coefficient for the coefficient of horizontal subgrade reaction
μ_m	:	allowable ductility ratio of a reinforced concrete columns in seismically-isolated bridge
μ_{Ri}	:	response ductility factor of the column representing the i-th design vibration unit
σ_v	:	total overburden pressure, (kN/m^2)
σ'_v	:	effective overburden pressure, (kN/m^2)
λ	:	ratio of coefficient of horizontal shear subgrade reaction to coefficient of vertical subgrade reaction; this coefficient can take the value λ = 1/3 ~ ¼
ϕ	:	resistance factor; a statistically based multiplier applied to nominal resistance specified in Sections 5, 6, 7, 8, 10, 11, and 12 of the *AASHTO LRFD Bridge Design Specifications*; flexural resistance factor for column, ϕ = 0.90 for columns with either spiral or tie reinforcement); fan-shaped angle by the two edges of a continuous girder of a curved bridge, (degrees)
φ	:	resistance factor for pile under extreme event limit state
τ_t	:	shear strength intensity of each layer, (kN/m^2)

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SECTION 3:
GENERAL REQUIREMENTS

3.1 APPLICABILITY OF SPECIFICATIONS

(1) These Specifications shall be taken to apply to the design and construction of conventional bridges to resist the effects of earthquake motion.

For non-conventional bridges and other types of construction (e.g. suspension bridges, cable stayed bridges, arch type bridges, and movable bridges), the DPWH shall require and approve appropriate provisions from the Designer. Moreover, for bridges not fully covered herein, the provisions of these Specifications may be applied, as augmented with additional design criteria where applicable, but with prior approval from the DPWH.

(2) The provisions of these Specifications shall be taken as the minimum requirements necessary to provide for structural stability and public safety. Additional provisions may be specified by the DPWH to achieve higher performance criteria for repairable and minimum damage attributed to essential or critical bridges. Where such additional requirements are specified, they shall be site or project specific and are tailored to a particular structure type. The DPWH may require, if necessary, the sophistication of design or the quality of materials and construction to be higher than the minimum requirements.

(3) Seismic effects for box culverts and buried structures need not be considered except where failure of the box culvert or the buried structures will affect the function of the bridge or where they cross active faults.

(4) The potential effects of unstable ground conditions (e.g. liquefaction, landslides and slope movements, and fault displacements) on the stability and function of the bridge should be considered.

(5) Detailed seismic structural analysis may not be required for any bridge in Seismic Performance Zone 1. Specific detailing requirements shall be applied for Seismic Performance Zone 1 and for single span bridges. For single-span bridges, minimum support length requirement shall apply according to Article 4.6.

(6) Other provisions not contained in these Specifications shall conform to requirements of the *AASHTO LRFD Bridge Design Specifications, 2012 or later edition*. However, the seismic design of bridges shall also conform to the requirements of the updated and revised DPWH Guide Specifications.

Commentary C3.1

Conventional bridges include those with slab, beam, box girder, or truss superstructures; have single- or multiple-column piers, wall-type piers, or pile-bent substructures; and are founded on shallow- or piled-footings, or shafts. Substructures for conventional bridges are also listed in Table 3.8.1-1. Nonconventional bridges include bridges with cable-stayed/cable-suspended superstructures, bridges with truss towers or hollow piers for substructures, and arch bridges. Single-span bridges shall be slab or girder bridges without piers and directly supported by abutments.

Since the DPWH is in the process of updating and revising its Design Guide Specifications, these Specifications shall comply with the other provisions and requirements of the *AASHTO LRFD Bridge Design Specifications, 2012 or later edition* for specific requirements not contained herein. However, once the DPWH Design Guide Specifications have been completed, these Specifications shall be used together with the DPWH Design Guide Specifications.

3.2 BRIDGE OPERATIONAL CLASSIFICATION

(1) For the purpose of seismic design, bridges shall be classified into one of the following three operational categories:

Table 3.2-1 Operational Classification of Bridges

Operational Classification (OC)	**Serviceability Performance**	**Description**
OC-I (Critical Bridges)	• Bridges that must remain open to all traffic after the Level 2 design earthquake, i.e. 1,000 year return period event. • Other bridges required by DPWH to be open to emergency vehicles and vehicles for security/defense purposes immediately after an earthquake larger than the Level 2 design earthquake (AASHTO recommends a 2,500-year return for larger earthquakes).	Important bridges that meet any of the following criteria: • Bridges that do not have detours or alternative bridge route (e.g. bridges that connect islands where no other alternative bridge exist), • Bridges on roads and highways considered to be part of the regional disaster prevention route, • Bridges with span ≥ 100m, • Non-conventional bridges or special bridge types such as suspension, cable stayed, arch, etc. • Other bridge forms such as double-deck bridges, overcrossings or overbridges that could cause secondary disaster on important bridges/structures when collapsed, • As specified by the DPWH or those having jurisdiction on the bridge.
OC-II (Essential Bridges)	• Bridges that should, as a minimum, be open to emergency vehicles and for security/defense purposes within a short period after the Level 2 design earthquake, i.e. 1,000 year return period event.	Bridges located along the following roads/highways: • Pan-Philippine Highway, • Expressways (Urban and Inter-urban expressways), • Major/Primary national arterial highways (North-South Backbone, East-West Lateral, Other Roads of Strategic Importance), • Provincial, City and Municipal roads in view of disaster prevention and traffic strategy. Additionally, bridges that meet any of the following criteria: • Bridges with detours greater than 25 kilometers • As specified by the DPWH or those having jurisdiction on the bridge
OC-III (Other Bridges)	• All other bridges not required to satisfy OC-I or OC-II performance.	• All other bridges not classified as OC-I or OC-II

The DPWH or those having jurisdiction shall classify the bridge into one of the above three operational categories.

(2) The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, considerations should be given to possible future changes in conditions and requirements.

Commentary C3.2

(1) Operational Classification II (OC-II or Essential Bridges) covers generally those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake, i.e., a Level 2 1,000-yr return period event. However, some bridges must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., more than 1,000-yr return period event. These bridges should be regarded as critical structures with Operational Classification OC-I.

Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge. In this case, critical bridges defined in Table 3.2.1, being major and important bridges, shall have higher strength and performance requirements than essential or other bridge classifications. In this case, critical bridges are required to be open to all traffic after the Level 2 design earthquake but in the event of an earthquake larger than the Level 2 design earthquake, critical bridges should at least be open to emergency vehicles and vehicles for defense purposes. For earthquakes larger than the design earthquake, AASHTO recommends earthquake with a 2,500-yr return period.

Since critical bridges requires higher strength and performance than essential and others, careful design and details shall be undertaken to ensure its performance under large earthquakes.

(2) Bridges are classified as to OC-I (Critical), OC-2 (Essential) and OC-II (Others), depending on the importance factors such as the road class, bridge functions and structural characteristics. In this regard, when classifying bridge importance in view of the roles expected in the regional disaster prevention plan, the following factors shall be taken into account:

(1) Role expected in the regional disaster prevention plan.

Consideration shall be given to the extent to which the bridge is necessary, as an emergency transport route, for post-earthquake event rescue and recovery activities. The DPWH shall clearly define which route shall be included under the disaster prevention plan.

(2) Possibility of a secondary disaster

Consideration shall be given to the extent to which damages to bridges (such as double-deck bridges and overcrossings) affect other important lifeline structures and facilities.

(3) Serviceability and availability of alternate routes

Consideration shall be given to the present traffic volume on the bridge and the availability of alternate routes when the bridge loses its pre-event functions. Since bridges play an important role for social and economic activities between communities and centers, the availability of alternate route shall be one factor to decide the bridge operational classification.

(4) Difficulty in recovering bridge function

Consideration shall be given to the duration and cost for recovering the function of a damaged bridge.

The DPWH Planning Service shall be responsible for identifying the operational classifications of bridges considering the above roles and functions of the roads and bridges.

3.3 SEISMIC PERFORMANCE REQUIREMENTS

3.3.1 General

(1) Bridges shall have the following three levels of seismic performance:

1) Seismic Performance Level 1 (SPL-1)

Performance level of a bridge to ensure its normal sound functions during an earthquake.

2) Seismic Performance Level 2 (SPL-2)

Performance level of a bridge to sustain limited damages during an earthquake and capable of recovery immediately for critical bridges and within a short period for essential bridges.

3) Seismic Performance Level 3 (SPL-3)

Performance level of a bridge to ensure safety against collapse during an earthquake.

(2) Bridges classified under the Operational Classification in Table 3.2-1 shall conform to the performance requirements given in Table 3.3.1-1 and the design earthquake ground motion.

Table 3.3.1-1 Earthquake Ground Motion and Seismic Performance of Bridges

Earthquake Ground Motion (EGM)	**Bridge Operational Classification**		
	OC-I (Critical Bridges)	**OC-II (Essential Bridges)**	**OC-III (Other Bridges)**
Level 1 (Small to moderate earthquakes which are highly probable during the bridge service life, 100-year return)	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*	SPL-1 *(Keep the bridge sound function; resist seismic forces within elastic limit)*
Level 2 (Large earthquakes with a 1,000-year return period)	SPL-2 *(Limited seismic damage and capable of immediately recovering bridge functions without structural repair)*	SPL-2 *(Limited seismic damage and capable of recovering bridge function with structural repair within short period)*	SPL-3 *(May suffer damage but should not cause collapse of bridge or any of its structural elements)*

(3) Bridges shall be designed to ensure that unseating of superstructures can be prevented, even if local structural failures of components (such as bearing supports, displacement limiting device, etc.) or large displacements may occur due to complicated structural behavior or ground deformation due to liquefaction, which were not expected in seismic design. Requirements for the unseating prevention system is specified under Section 7 of these Specifications.

(4) When verifying the seismic performance of bridges, the limit state of each structural member or component shall be appropriately determined in accordance with the limit states of the bridge as specified in Articles 3.3.2 to 3.3.4 of this Section.

Commentary C3.3.1

(1) The seismic performance of bridges as a goal in seismic design is classified into three levels in view of safety, serviceability and repairability. ***Safety*** implies performance to avoid loss of life due to

collapse or unseating of the superstructure during an earthquake. ***Serviceability*** means that the bridge is capable of keeping its bridge functions such as fundamental transportation function, the role as evacuation routes and emergency routes for rescue, first aid, medical services, firefighting and transporting emergency goods to refugees. ***Repairability*** denotes capability to repair seismic damages. Bridges that are designed and detailed in accordance with these provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

Seismic Performance Level 1 (SPL-1) given as "Performance level of a bridge to ensure its normal sound functions during an earthquake", implies that the bridge shall be restrained safely from unseating, no emergency repair work is needed to recover the functions soon after the earthquake, and repair work which may take long time can be easily conducted. This indicates full access to all traffic immediately following an earthquake.

Seismic Performance Level 2 (SPL-2) given as "Performance level of a bridge to sustain limited damages during an earthquake and capable of recovery within a short period", ensures not only safety in unseating prevention but also capability to recover the bridge functions soon after the event and repairability through comparatively easy, long-term repair work.

Seismic Performance Level 3 (SPL-3) given as "Performance level of a bridge to ensure no critical damage during an earthquake", implies that safety against collapse and unseating is ensured but does not cover the function necessary for serviceability and repairability for seismic design.

Table C3.3.1-1 summarizes the items for Seismic Performance Levels 1 to 3, considering safety, serviceability and repairability for seismic design.

Table C3.3.1-1 Seismic Performance of Bridges

Seismic Performance	**Seismic Safety Design**	**Seismic Serviceability Design**	**Seismic Repairability Design**	
			Emergency Repairability	**Permanent Repairability**
Seismic Performance Level 1 (SPL-1): *Keeping the sound function of bridges*	Ensure safety against girder unseating; resist earthquake within elastic range	Ensure normal bridge functions	No repair work is needed to recover bridge functions	Only easy and minor repair works are needed
Seismic Performance Level 2 (SPL-2): *Limited damages and recovery*	Ensure safety against collapse and girder unseating	Capable of recovering functions within a short period after the earthquake event	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair work
Seismic Performance Level 3 (SPL-3): *No critical damages*	Ensure safety against collapse and girder unseating	-	-	-

Figure C3.3.1-1 shows the relationships between lateral load-displacement curve and the seismic performance level.

SPL-1 specifies prevention of structural damage on members by confining the response within the elastic limits of the members. In this case, small to moderate earthquakes have to be resisted by the structural members at the elastic limit so that the bridge is immediately open to traffic after the earthquake without the need for substantial repairs. The bridge normal function is maintained after the earthquake and if damage did occur, it must be minor which can easily be repaired without disrupting the bridge function.

SPL-2 indicates the repairable limit of the members by limiting the damage to within recoverable level. Under this performance level, the bridge function can easily be recovered within a short period after an earthquake event through emergency repair works. Moreover, permanent repairs for damages can be done easily. Under SPL-2, bridge components are allowed to behave inelastically to such extent that its function can be easily recovered without loss of capacity and serviceability.

SPL-3 specifies ductility capacity limit to prevent critical damages on members and thus prevent bridge collapse. Under this performance level, bridge members and components are allowed to undergo inelastic behavior during large earthquakes but should not cause collapse of any or all of its main structural components. The extent of damages in this level may require replacement of the components or the bridge itself.

Both SPL-2 and SPL-3 may approach post-elastic, non-linear structural behavior.

In all SPL cases, bridge collapse and girder unseating should be prevented.

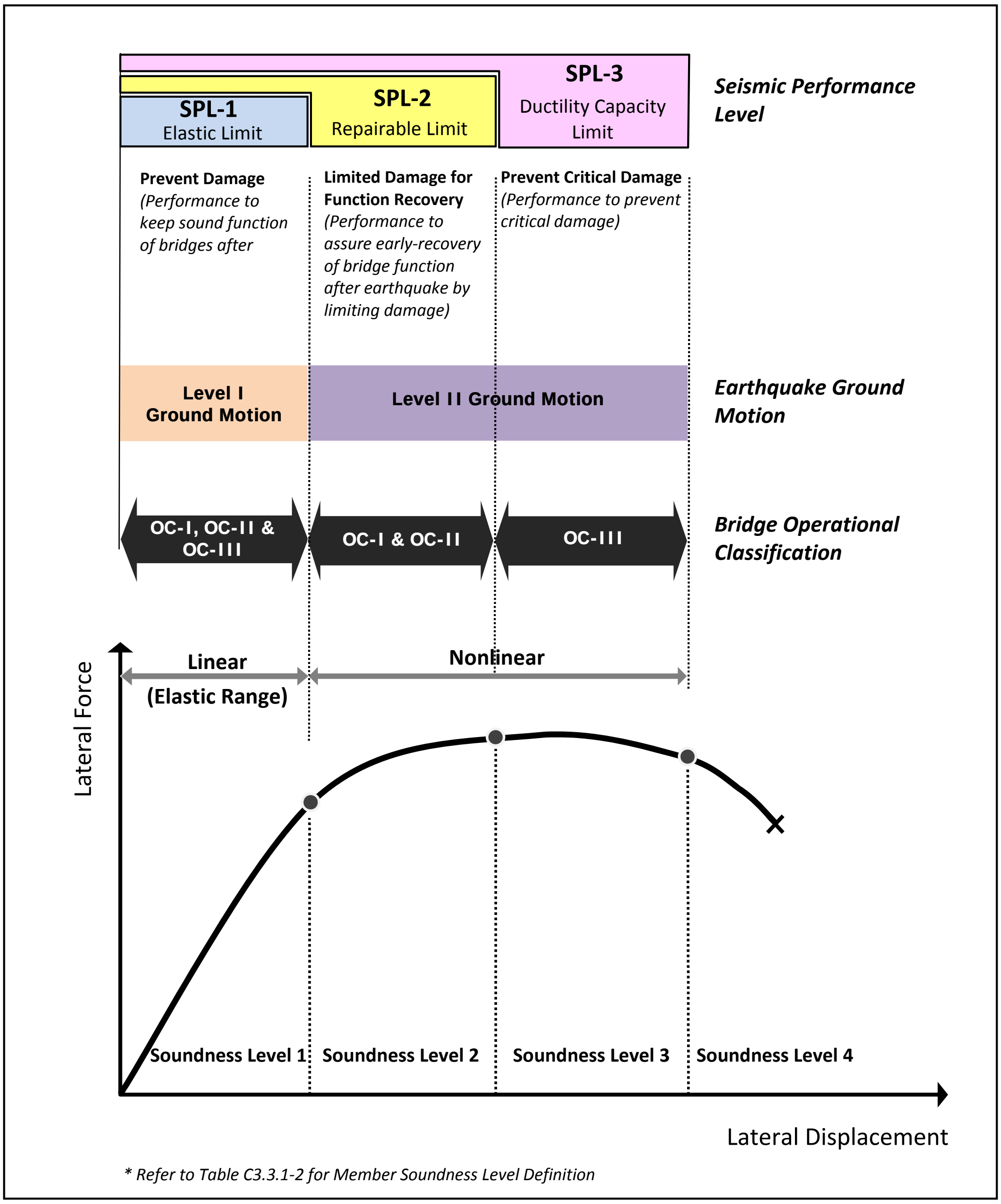


Figure C3.3.1-1 Relationship between Lateral Load-Displacement Curve, Seismic Performance Level, Earthquake Ground Motion and Operational Classification

The AASHTO LRFD traditional procedure to seismic design of bridges, which is adopted in these Specifications, utilizes the force-based approach where structural damage is controlled by an assignment of a certain level of strength derived by application of the response modification (R-factor). The R-factors generalizes the ductility capacity of the members where columns are assumed to deform inelastically when seismic forces exceed their design level. This is taken by dividing the elastically computed force effects by an appropriate force-reduction factor (R-factor). In this case, the structure, particularly columns, should have enough ductility to be able to deform inelastically to the deformations caused by large earthquakes without loss of post-yield strength. The force-reduction factors are then specified to determine the inelastic deformation demands on the bridge members when the design earthquake occurs. Since these Specifications adhere to the AASHTO force-based approach to seismic design, it is assumed that seismic performances of bridges required under the different bridge operational classifications are addressed by the corresponding R-factors.

In order to assure bridge seismic performance during an earthquake, all bridges and their foundations and structural elements shall be capable of resisting the seismic demand forces and achieving the anticipated displacements required under the objective seismic performance level. In this sense, earthquake resisting systems (ERS), as shown in *Appendix 1A*, shall be clearly identified and selected to achieve the requirements of Article 3.3.1(1) seismic performance levels. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase, or the type, size, and location phase, or the design alternative phase of the project.

Considering seismic repairability after an earthquake, bridge members and structural elements shall be capable of maintaining a structural soundness consistent with the seismic performance level requirements. The anticipated soundness level of the ERS elements and bridge members should be defined during the design process as indicated in Table C3.3.1-2.

Table C3.3.1-2 Member Soundness Level

	Soundness Level 1	**Soundness Level 2**	**Soundness Level 3**	**Soundness Level 4**
Member Condition	No damage/elastic range	Limited damage/Still possess remaining strength and displacement capacity	Severe damage/Near strength and displacement capacity	Beyond strength and ductility
Member Restoration and Strengthening	No restoration	Minor/easy restoration/permanent repair works	Restoration and strengthening is possible/May require replacement	Member replacement is necessary

(2) The seismic performance requirements for bridges at different operational classification shall be based on the design earthquake ground motion which is given in Article 3.4. The relationship between the operational classification and earthquake ground motion and the load-displacement curve is illustrated in Figure C3.3.1-1 where all bridges (OC-I to OC-III) are expected to respond within the elastic limit at Level 1 earthquake ground motion (small to moderate earthquakes). On the other hand, critical and essential bridges (OC-I and OC-II, respectively) should behave within the post-elastic repairable limit while other bridges (OC-III) should not exhibit critical damage or collapse under Level 1 ground motion (large earthquake).

All bridges (OC I-III) are expected to perform its normal function or be open to all traffic under Level 1 earthquake ground motion which has high probability of occurrence during the service life of the bridge. In this case, Level 1 earthquake shall be resisted within the elastic range of the structural components without significant damage.

However, under a large earthquake with low probability of occurrence during the bridge service life or Level 2 earthquake ground motion, bridges classified as OC-III should not collapse and should have no critical damage. Moreover, bridges classified under OC-I and OC-II should have limited damage and are capable of recovering the bridge functions within a short period. In this case, OC-I bridges shall be open to all traffic under Level 2 earthquake and as a minimum, be open immediately to emergency vehicles and for security/defense purposes after an earthquake greater than Level 2 design earthquake. On the other hand, OC-II bridges should, as a minimum, be open to emergency vehicles and for security/defense purposes within a short period after the Level 2 design earthquake.

(3) In the event that local structural failures of components (such bearing supports, displacement limiting device, etc.) or large displacements occur due to complicated structural behavior or ground deformation due to liquefaction which were not expected during seismic design, safety measures to prevent superstructures from unseating should be ensured.

Although the earthquake ground motions similar to that of the Hyogo-ken Nanbu Earthquake which is considered one of the most devastating earthquakes in the world have been recorded, similar or stronger earthquake ground motion may be generated in the future. This is because of the mechanism of generating strong earthquake ground motions and the properties of active faults causing future earthquakes are still uncertain. In addition, since the time range for observing strong earthquake is short and observation records of such strong earthquake ground motions are lacking in the Philippines, an accurate estimation of earthquake ground motion is difficult to incorporate in the seismic design of bridges. Moreover, unexpected forces, displacements and deformations may generate in bridges due to the failure of the surrounding ground or the unexpected complicated behavior of the structural members. In this case, safety measures against unseating shall be ensured as much as possible. In the Kocaeli, Turkey Earthquake of 1999 and the Chichi, Taiwan Earthquake of 1999, the superstructure fell down due to extremely large fault displacements of 5 to 10m.

In order to directly incorporate the characteristics of individual active faults into seismic design, locations and active periods of the faults, and the ground surface dislocations caused by the faults should be identified. Although PHIVOLCS is actively studying and mapping of active faults, many points still remain unclear. For instance, the fault source that generated the 7.2 magnitude earthquake in Bohol on October 15, 2013 was not identified previously by PHIVOLCS. It is then necessary to carefully examine these topics through further research.

In view of the uncertainty and the unexpected phenomena that may occur, prevention of superstructures from unseating shall be one of the goals in seismic design of bridges. However, it is still difficult to design a completely safe bridge against any earthquake ground motion or extremely large fault displacements. In preparation for such earthquakes, it is important to enhance redundancy of highway networks, and to develop preparedness systems and techniques that will allow earlier recovery.

3.3.2 Requirements for Seismic Performance Level 1 (SPL-1)

The limit state of the bridge for Seismic Performance Level 1 shall ensure that the mechanical properties of the substructure and superstructure members and components are maintained within the elastic range and that these members will perform without or with minimal damage. The bridge members and components to be verified under this performance level shall include pier columns or piers walls, abutments, bearings, expansion joints and unseating prevention system devices.

Commentary C3.3.2

The limit states of the members and components of the bridge for Seismic Performance Level 1 shall be properly established so that the bridge functions before an earthquake is ensured after an earthquake and damages to any structural member is kept to minimal. In this regard, the limit state of each structural member for Seismic Performance Level 1 shall be that the mechanical properties are maintained within the elastic range. Foundation shall also behave under the elastic range.

Structural members are expected not to have any damage under this seismic performance level. Any damage that does occur shall be minimal without effect on structural behavior and can be repaired easily and quickly.

3.3.3 Requirements for Seismic Performance Level 2 (SPL-2)

(1) The limit state of the bridge for Seismic Performance Level 2 shall ensure that only the structural member where formation of plastic behavior (plastic hinging) are expected shall be allowed to deform plastically within the range of easy recovery of bridge function.

(2) The structural member in which formation of plastic hinges are allowed shall be properly selected so that reliable energy absorption mechanism is observed and that recovery is ensured in a short period of time. Plastic hinges are allowed to form at the bottom of the pier columns for single column piers and at the top and bottom for rigid frame piers or piers rigidly connected with the superstructure, as shown in Figure C3.3.3-1.

(3) These structural members with plasticity shall be properly combined with other structural members in accordance with the anticipated structural characteristics and behavior of the bridge. The limit state of each member under the combination shall be appropriately determined to ensure structural performance under earthquake loading.

Commentary C3.3.3

(1) This limit states of the bridge for Seismic Performance Level 2 shall ensure that only the structural member in which the formation of plastic behavior or plastic hinges during an earthquake are allowed deforms plastically within the range of easy recovery of function. Members with and without plastic behavior during large earthquake shall be identified clearly.

(2) The structural member in which reliable energy absorption, easy inspection of the damage and recovery in a short period are ensured due to its plastic behavior, mainly corresponds to bridge piers.

Since superstructures are directly related with traffic serviceability, formation of plastic hinges in superstructures are basically not recommended from serviceability point of view in seismic design. However, for bridges with special structure such as rigid-frame bridges in which effects of earthquake predominate the design of superstructure, and long span bridges, superstructures without plastic behavior during an earthquake will possibly lead to an extremely uneconomical design. On the other hand, in the design of a prestressed concrete bridge superstructure rigidly connected to the piers, excessive reinforcing steel bars for avoiding plasticity of the superstructure may cause a big loss in prestressing forces. In this case, secondary plasticity can occur in the superstructure, in addition to the assumption that plastic hinges are principally formed in the piers. Here, the secondary plasticity denotes for that the structural members with plastic behavior play a great role in energy absorption where local plastic hinges are reached in the seismic response.

Seismic isolation bearings are members with the mechanism having nonlinear hysteretic characteristics and capable of absorbing energy with deformation. When such kinds of seismic isolation bearings are applied in seismically isolated bridges, the limit states of each bridge member shall be properly determined in consideration of the plastic behavior of the bearings that will ensure energy absorption by these bearings. Instead of expressing the mechanical properties for the seismic isolation bearings in terms of plastic behavior, nonlinear behavior shall be used.

Since the foundations are likely to play a role in improving the ability of energy absorption, some damages may happen in the process of energy absorption as the foundations deform. In this case, it is not easy to find the damage of the foundations, and the recovery becomes serious and very difficult. Although the foundation can be designed to allow generation of secondary plasticity without considering principal plastic behavior, it is recommended in this Specification that foundations behave elastically under earthquake forces.

For non-conventional bridges such as arch bridges, cable-stayed bridges or suspension bridges, structural members with plastic behavior shall be properly selected according to the structural behavior of each bridge. In addition, the limit states of these members shall also be appropriately set based on the limit states of entire bridge system. On the other hand, the structural members such as the arch ribs in arch bridges or the main pylon/tower of cable-stayed bridges which play an important role in keeping the dynamic behavior and stability of the entire bridge system and those members subjected to large axial forces in the normal conditions which may vary during an earthquake, shall be carefully investigated and designed within the allowable plastic ranges since the dynamic behavior of such members are still unclear up to now.

(3) Structural members with plastic behavior shall be properly combined in accordance with the bridge types, expected behavior under earthquake and the limit state of entire bridge system. As explained earlier, the structural members in piers, seismic isolation bearings and foundations (in some cases) can absorb energy because of their plastic behavior or nonlinear properties. Table C3.3.3-1 shows some examples on how to combine the members with plastic behavior or nonlinear properties and how to determine the limit states of the combined structural members so as to ensure the Seismic Performance Level for an entire bridge system. However, the limit state of the expansion joints connecting the ends of the superstructure are not indicated so that traffic can be kept passable through simple recovery within a short period of time in case such damages occur.

Another way to ensure reliable energy absorption is using a structure consisting of several members that work simultaneously. In this case, its dynamic behavior is likely to get more complicated during a large earthquake, so that further investigation on response characteristics of this kind of structure is necessary. At the present stage, selecting structural members with plastic behavior or nonlinear properties from either the pier, seismic isolation bearing of a seismically isolated bridge or foundation is desirable to get a more reliable energy absorption mechanism, as shown in Figure C3.3.3-1.

Table C3.3.3-1 shows the limit state of each member with plastic behavior or nonlinear properties corresponding to their combination as follows:

1) When considering the formation of plastic hinges in piers

The limit state of the piers with plastic behavior shall be the limit state in which the damages in the piers can be easily recovered. In other words, for purpose of ensuring rapid recovery in bridge functions, the limit state denotes the state just before reduction of horizontal strength begins, or the state that residual displacement recovery is difficult.

In order to ensure that plastic hinges develop mainly in the piers, the limit states of other members including abutments, bearing support system and superstructures shall be that the mechanical properties of the members are within the elastic range. Bearing support systems like rubber bearing supports shall be capable of absorbing the deformations considering its proven mechanical properties to be within the range without damage.

Basically, the mechanical properties of the foundations shall be kept within the elastic range. However, when it is unavoidable to keep the foundation within the elastic range due to liquefaction and lateral spreading, the secondary plastic behavior may be allowed in order to prevent the foundation from generating damages in the whole foundation that will be difficult to recover. This is based on the consideration that some of the foundation members may deform plastically other than the nonlinear effects of the foundations. Since footings or pile caps play the role of transferring smoothly the seismic forces acting on the piers to foundations, the limit state of footing or pile cap shall be that the mechanical properties are kept within the elastic ranges.

2) When considering the formation of plastic hinges in piers and superstructures

When considering the formation of plastic hinges in the piers and superstructures, the limit state of the piers shall be that the bridge functions can be easily recovered as mentioned in Item 1) above.

The limit state of the superstructures were generally the state that allows only secondary plastic behavior after primary plastic hinges developed in the piers. However, since pavements and handrails are in the superstructure that is directly subjected to live loads and that space for recovery works is limited, it is preferable to define the limit state to be a state capable of keeping the serviceability with only slight damages even if plasticity is considered.

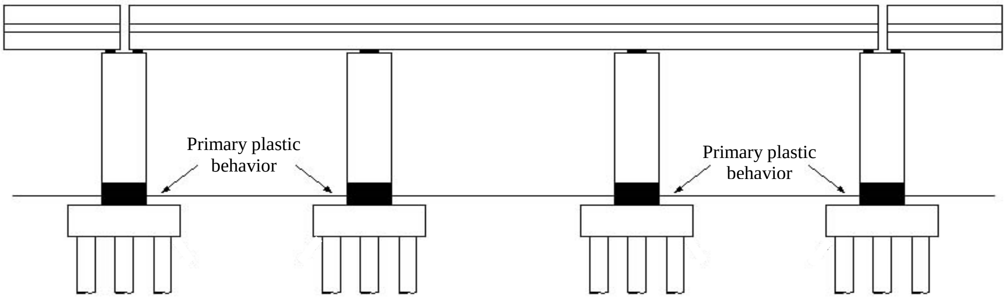
In case that the superstructure is connected directly to the pier like rigid-frame bridge, excessive bending strength on top of the pier may lead to a larger seismic force being transferred to the superstructure. Thus, the section on top of the pier should be carefully designed so that plastic hinges will form in the pier first and any failure occurring in the superstructure can be properly controlled. In addition, special attention should be considered in the design details since the secondary plastic behavior of a prestressed concrete superstructure in a rigid-frame bridge, as shown in Figure 3.3.3-1(e), may easily form near the sections with zero bending moment (inflection point) under normal load conditions,. However, since the plastic behavior of steel superstructure remains unclear, careful investigation on allowable ranges of the plastic behavior is required if the secondary plastic behavior is included.

3) When considering the formation of plastic hinges in foundations

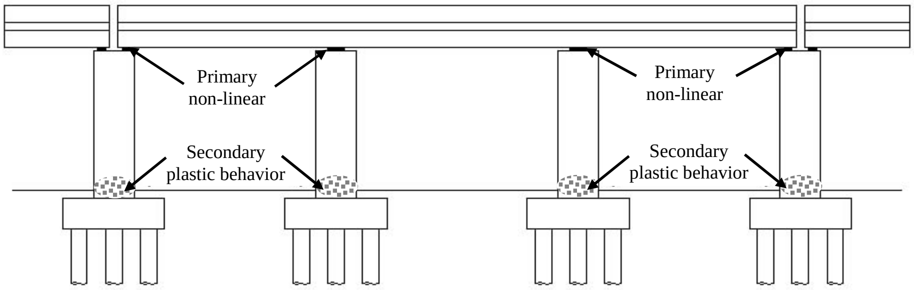
In this case, the limit state shall be the state without excessive deformation or damage that may affect recovery works. When considering energy absorption of the foundation, the dynamic behavior of the foundation shall be kept within the range of estimated subgrade reactions without generating large deformation of the foundation that could affect the superstructure behavior and create damages in the superstructure.

Under this condition, the limit states of each members in the piers, abutments, bearing support system or superstructure shall be the state that the mechanical properties will be within the elastic ranges.

However, these Specifications recommends that foundations be kept within the elastic range of properties unless liquefaction and lateral spreading results to impractically large foundation structures. In this case, formation of secondary plastic hinges may be allowed at the foundation but careful check shall be done to limit large structural deformation caused by this secondary hinging.

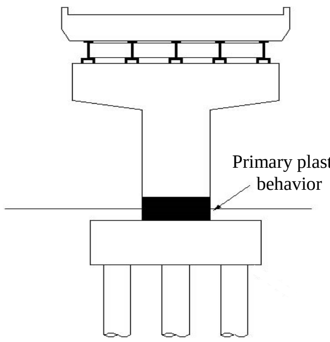


(a) Single Column Pier with Plastic Behavior
(Longitudinal Direction)

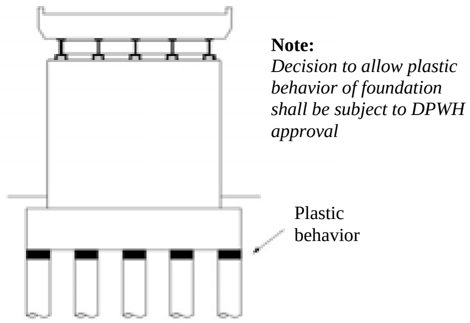


Seismically-Isolated Bridge

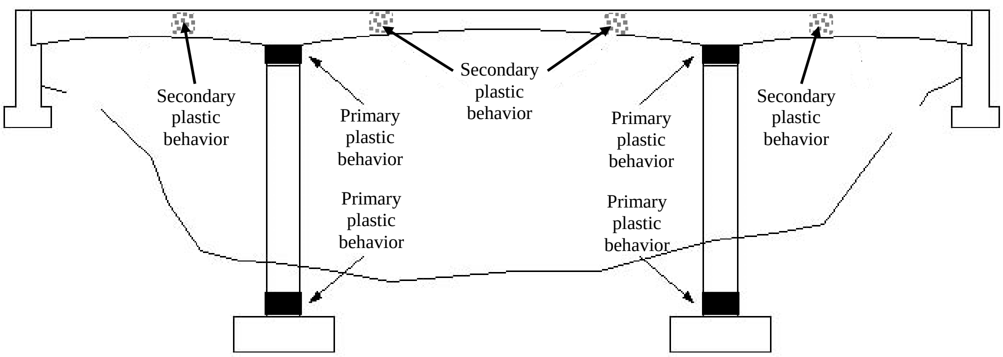
(b) Seismic Isolation Bearings with Consideration of Non-linearity



(c) Single Column Pier with Plastic Behavior
(Transverse Direction)



(d) Foundations with Plastic Behavior
(Pier Wall in Transverse Direction)



(e) Case of Considering Plasticity in Piers and Superstructure
(Rigid-Frame Bridge in Longitudinal Direction)

Figure 3.3.3-1 Combination Examples of Members with Consideration of Plasticity or Non-Linearity

4) When considering both plasticity of piers and non-linearity of seismic isolation bearings

For a seismically-isolated bridge, the limit state of the seismic isolation bearing shall be that only secondary plastic behavior of piers is allowed and the energy absorption capability of the bearings are ensured. This is because piers are expected to participate to some extent in the energy absorption for the entire bridge system in addition to the seismic isolation bearings. However, excessive share in the energy absorption of the piers may cause the damping properties of the seismic isolation bearings due to be ineffective. In such condition, only secondary plastic behavior shall be allowed for the piers so that the seismic isolation bearings will have a principal share in the energy absorption instead of the piers and thus reduce pier damage.

Table C3.3.3-1 Combination Examples of Members Considering Plasticity (Non-linearity) and Limit States of Each Members (For Seismic Performance Level 2)

Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers	Piers and Superstructures[2]	Foundations[1] (when unavoidable due to liquefaction and lateral spreading)	Seismic Isolation Bearings and Piers
Piers	Plastic hinging of pier within the range of easy recovery of bridge function	Plastic hinging of pier within the range of easy recovery of bridge function	Mechanical properties to be kept within the elastic range	Allow secondary plastic behavior
Abutments	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties to be kept within the elastic ranges
Bearings Support System	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Ensure reliable energy absorption through seismic isolation bearings
Superstructures	Mechanical properties within the elastic range	May allow secondary plastic behavior	Mechanical properties within the elastic range	Mechanical properties within the elastic range
Foundations	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Without excessive deformation or damage to disturb recovery works	Mechanical properties within the elastic range
Footings	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range
Application Examples	Deck girder bridges other than seismically isolated bridges	Rigid-frame bridges	Piers with sufficient strength or cases with unavoidable effects of liquefaction	Seismically isolated bridges

Note: [1] *Formation of plastic hinge are basically not allowed for foundation members. However, in cases where it is unavoidable due liquefaction and lateral spreading, secondary plastic hinging may be allowed when design of foundation under elastic range results in impractically large structures.*

[2] *In the same manner as foundation, formation of plastic hinging for superstructure shall be avoided. However, for piers rigidly connected to the superstructure, such as prestressed concrete girders, the possibility of formation of secondary plastic hinges in the superstructure shall be investigated if it will result in a better structural response mechanism to large earthquakes.*

5) When plastic behavior of structural members can be neglected

For reinforced concrete piers with shear failure or bridges without formation of plastic hinges in any member due to structural reason, the limit state of the members for Seismic Performance Level

2 shall be that the response of the members be kept within the elastic ranges without the occurrence of brittle failure. Here, the limit state represent the state when the horizontal forces acting on the pier will reach the shear strength for a reinforced concrete pier with shear failure, and that bending moments will reach the yield moment capacity for a reinforced concrete pier with flexural failure.

3.3.4 Requirements for Seismic Performance Level 3 (SPL-3)

(1) The limit state of the bridge for Seismic Performance Level 3 shall ensure that only the structural member where formation of plastic behavior (plastic hinging) are expected shall be allowed to deform plastically within the range of the member ductility.

(2) The structural members in which the generations of plastic hinges are allowed shall be selected so that a reliable energy absorption mechanism is ensured.

(3) These structural members with plasticity shall be properly combined with other structural members in accordance with the anticipated structural characteristics and behavior of the bridge. The limit state of each member under the combination shall be appropriately determined to ensure structural performance under earthquake loading.

Commentary C3.3.4

(1) The limit states of bridges for Seismic Performance Level 3 shall be properly established so that only the structural members in which the development of plastic behavior are allowed to deform plastically within the range of ductility limit. Under this performance level, members where the formation of plastic hinges are allowed shall be distinguished first from those which plastic hinges are not allowed. Then, only those members which are allowed to develop plastic hinges shall be forced to deform plastically during an earthquake which will prevent fatal damages to the bridge.

(2) For seismically-isolated bridges, the seismic isolation bearings are generally taken as the members that are capable of absorbing energy during an earthquake, similar to the limit state of bridges for Seismic Performance Level 2 in Article 3.3.3.

(3) Although the piers of seismically-isolated bridges can also serve as a member capable of absorbing energy, their combination shall be properly determined depending on the bridge type and the limit state of entire bridges specified in Item (1) above. Table C3.3.4-1 illustrates the example combinations of limit states for members to ensure the Seismic Performance Level 3 for the entire bridge system, and the limit states of each member corresponding to the combination of members with plastic or nonlinear behavior, as explained below:

1) When considering the formation of plastic hinges in piers

In this case, the limit state of the pier is defined as a state when the horizontal strength of the pier starts to reduce rapidly. The pier generally can keep its ductility within an allowable range at the state just before it loses its horizontal force resistance.

The limit states of the members such as abutments, bearings supports system, superstructures, foundations and footings are defined as the same as that for Seismic Performance Level 2.

2) When considering the formation of plastic hinges in piers and superstructures

For the same reasons stated in 1) above, the limit state of the pier is defined as a state when the horizontal strength of the pier starts to reduce rapidly. Since researches on ductility of superstructures remain insufficient, how to quantify the magnitude of superstructure plasticity is still unclear in case of Seismic Performance Level 3. Therefore, limit state of the superstructure can be established just like that for Seismic Performance Level 2.

In addition, the limit states for members of other bridge components can be determined on the basis of Table C3.3.4-1 similar to Article 3.3.3-1.

Table C3.3.4-1 Combination Examples of Members with Consideration of Plasticity (Non-linearity) and Limit States of Each Members (For Seismic Performance Level 3)

Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers	Piers and Superstructures[2)]	Foundations[1)] (when unavoidable due to liquefaction and lateral spreading)	Seismic Isolation Bearings and Piers
Piers	Horizontal strength of piers starts to reduce rapidly	Horizontal strength of piers starts to reduce rapidly	Mechanical properties within the elastic range	Allow secondary plastic behavior
Abutments	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range
Bearings Support System	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Ensuring reliable energy absorption by seismic isolation bearings
Superstructures	Mechanical properties within the elastic range	Allow secondary plastic behavior	Mechanical properties within the elastic range	Mechanical properties within the elastic range
Foundations	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Without excessive deformation or damage to disturb recovery works	Mechanical properties within the elastic range
Footings	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range	Mechanical properties within the elastic range
Application Examples	Deck bridges other than seismically-isolated bridges	Rigid-frame bridges	Piers with sufficient strength or cases with unavoidable effects of liquefaction	Seismically-isolated bridges

Note: *1) Formation of plastic hinge are basically not allowed for foundation members. However, in cases where it is unavoidable due liquefaction and lateral spreading, secondary plastic hinging may be allowed when design of foundation under elastic range results in impractically large structures.*

2) In the same manner as foundation, formation of plastic hinging for superstructure shall be avoided. However, for piers rigidly connected to the superstructure, such as prestressed concrete girders, the possibility of formation of secondary plastic hinges in the superstructure shall be investigated if it will result in a better structural response mechanism to large earthquakes.

3) When considering the formation of plastic hinges in foundations

Since the dynamic behavior of a foundation during large earthquakes remains unclear when severe damage generates in the foundation, the limit state of the foundation can be established in a similar manner to that for Seismic Performance Level 2. In this case, the limit state of other members such as piers, abutments, bearing supports or superstructures can be determined in such a way that their mechanical properties could be kept within the elastic ranges.

4) When considering both plasticity of piers and non-linearity of seismic isolation bearings

To avoid loss of the required resistance due to earthquake effects as the range of plasticity in piers get bigger than that in Seismic Performance Level 2, the limit state of seismically-isolated bridges can be defined just like the case for Seismic Performance Level 2.

5) When plastic behavior of structural members can be neglected

For bridges with reinforced concrete piers with shear failure mode, or bridges in which no plasticity are allowed in any members due to structural reasons, the limit states shall be established the same as that specified for Seismic Performance Level 2 in Article 3.3.3-1.

3.4 SEISMIC HAZARD

(1) The seismic hazard at a bridge site shall be characterized by the acceleration response spectrum for the site and the site factors for the relevant ground types (site class).

(2) The acceleration spectrum shall be determined using either the General Procedure specified in Article 3.4.1 or the Site Specific Procedure specified in Article 3.4.2.

(3) A Site-Specific Procedure shall be used, as may be required by DPWH, for very important bridges in close proximity with an active fault certified by PHIVOLCS.

(4) If time histories of ground acceleration are used to characterize the seismic hazard for the site, they shall be determined in accordance with Article 4.3.4.2 of these Specifications.

Commentary C3.4

The seismic hazard at the bridge site is basically characterized by the acceleration response spectrum at the site which can be determined by either the General Procedure given in Article 3.4.1 or the site specific procedure outlined in Article 3.4.2 of this Section. Basically, the General Procedure is used to determine the seismic hazard in all operational class categories.

However, the DPWH may require a site specific procedure when a very important bridge is located near an active fault. Other factors including ground condition at the site may require the use of site specific procedure.

3.4.1 General Procedure

(1) The General Procedure shall use the peak ground acceleration coefficient (PGA) and the short- and long-period spectral acceleration coefficients (S_S and S_1 respectively) to calculate the design response spectrum as specified in Article 3.6.

The values of PGA, S_S and S_1 shall be determined from the acceleration coefficient contour maps of Figures 3.4.1-1 to 3.4.1-3 for the Level 1 Earthquake Ground Motion and Figures 3.4.1-4 to 3.4.1-6 for Level 2 Earthquake Ground Motion of this Section for the entire Philippine archipelago and from Appendix 3A and 3B for the regional level acceleration coefficient contour maps as appropriate, or from site specific ground motion maps approved by the DPWH or the Owner.

(2) For sites located between two contour lines, the higher value of the two contour line shall be taken as the coefficient value.

(3) The effect of ground type (site class) on the seismic hazard shall be as specified in Article 3.5.

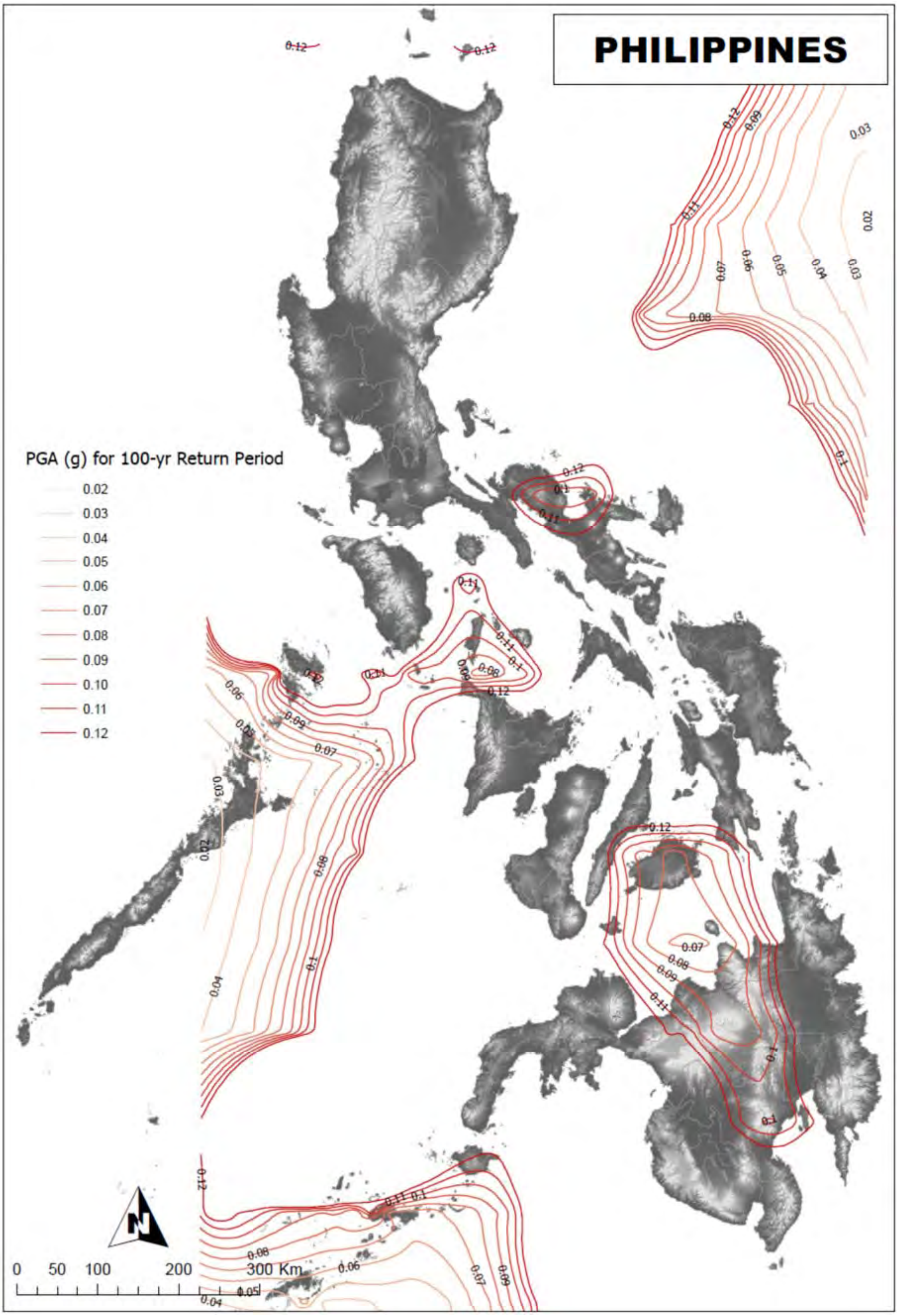


Figure 3.4.1-1 Horizontal Peak Ground Acceleration Coefficient (*PGA*) with 53% percent Probability of Exceedance in 75 Years (Approximately 100-year Return Period) for Level 1 Earthquake Ground motion

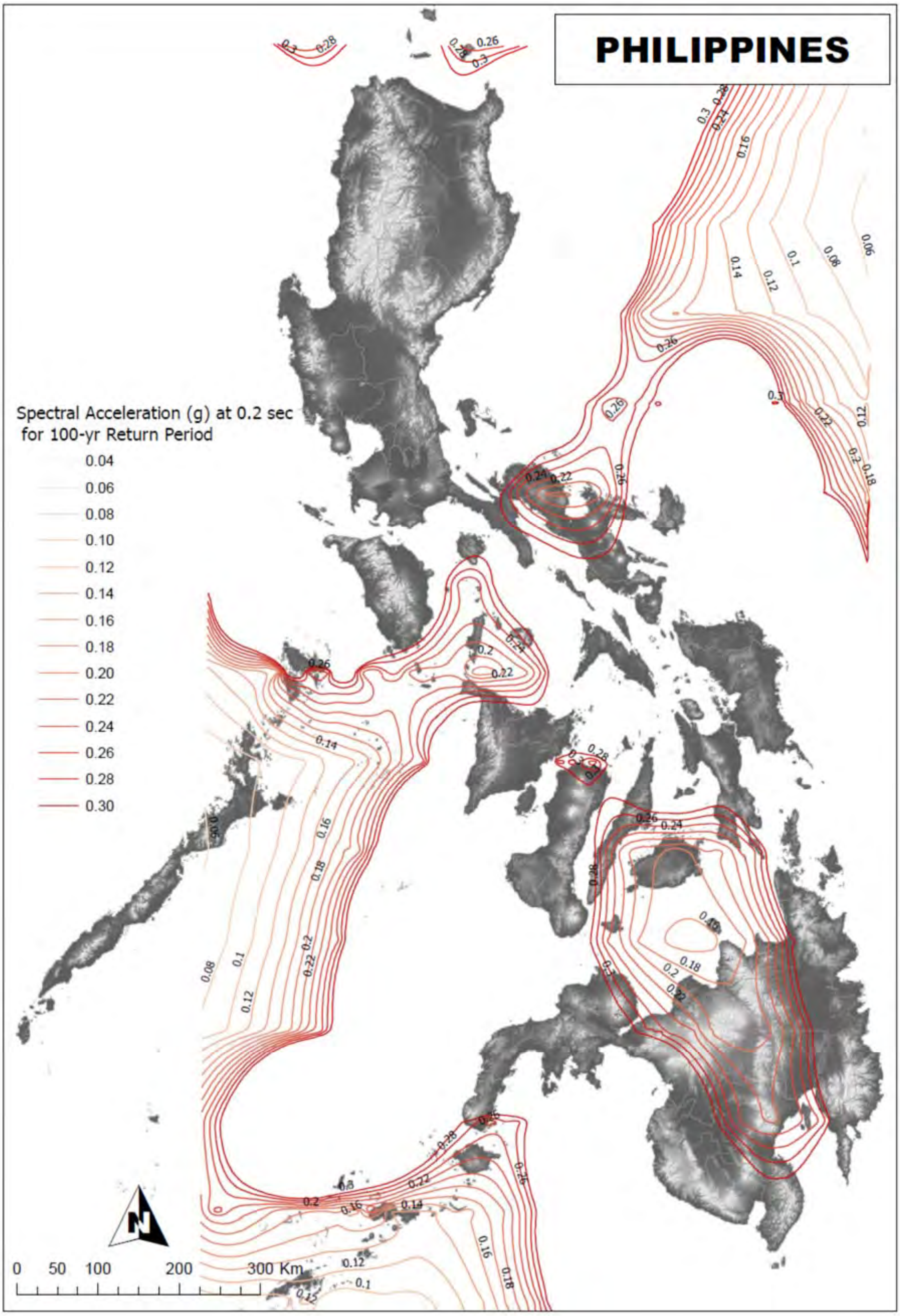


Figure 3.4.1-2 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_s) with 53% percent Probability of Exceedance in 75 Years (Approximately 100-year Return Period) for Level 1 Earthquake Ground Motion

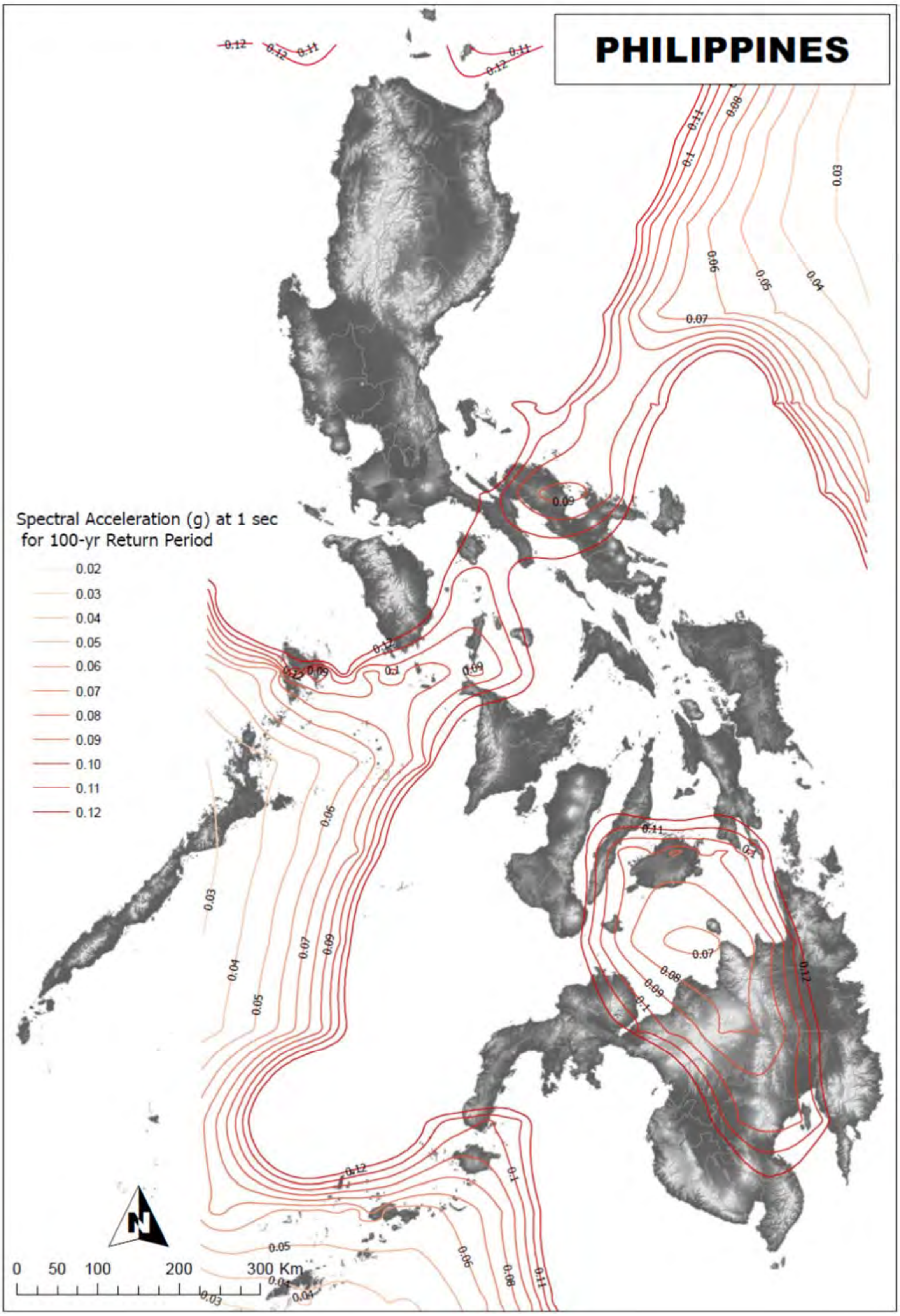


Figure 3.4.1-3 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) with 53% percent Probability of Exceedance in 75 Years (Approximately 100-year Return Period) for Level 1 Earthquake Ground motion

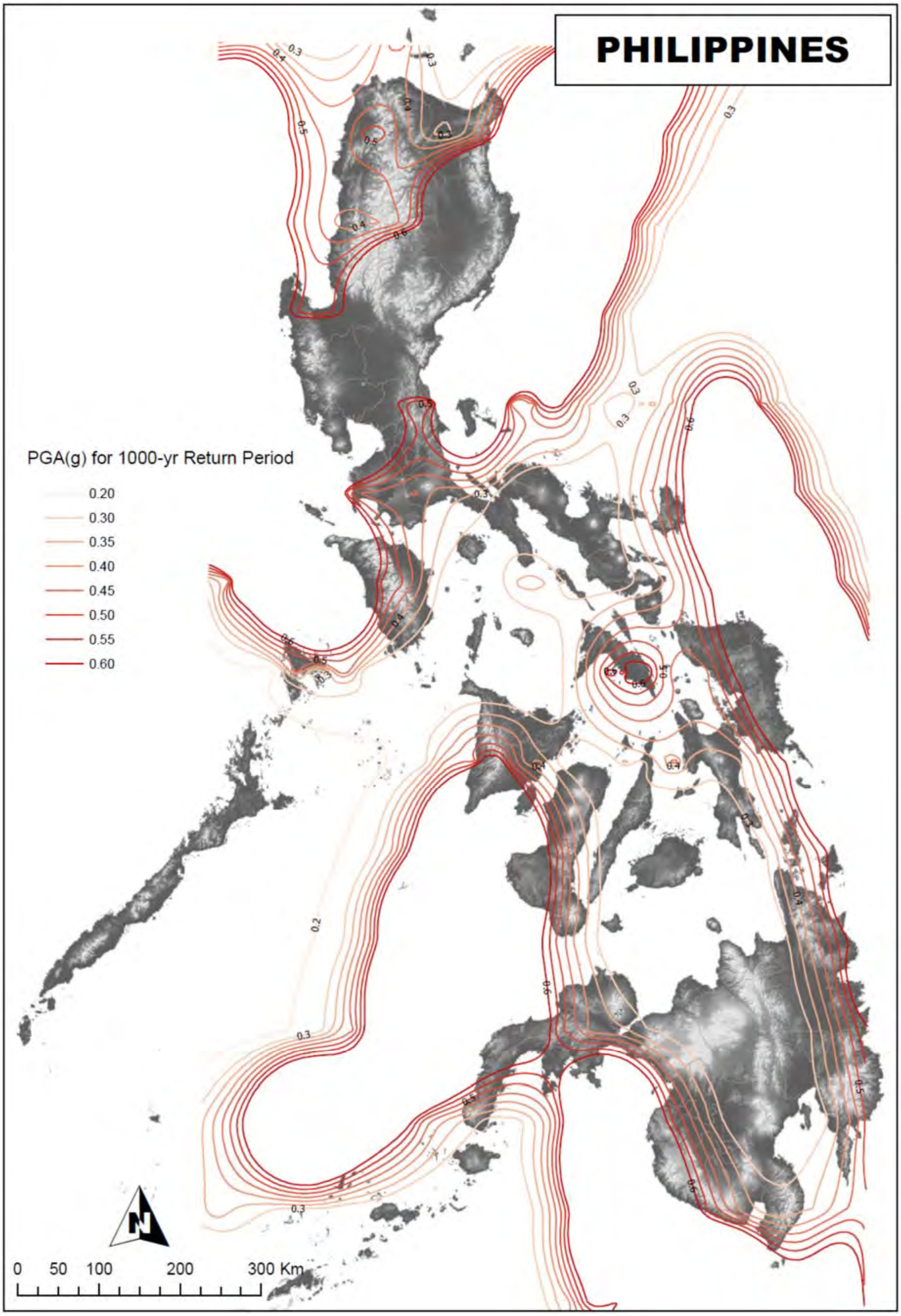


Figure 3.4.1-4 Horizontal Peak Ground Acceleration Coefficient (*PGA*) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground motion

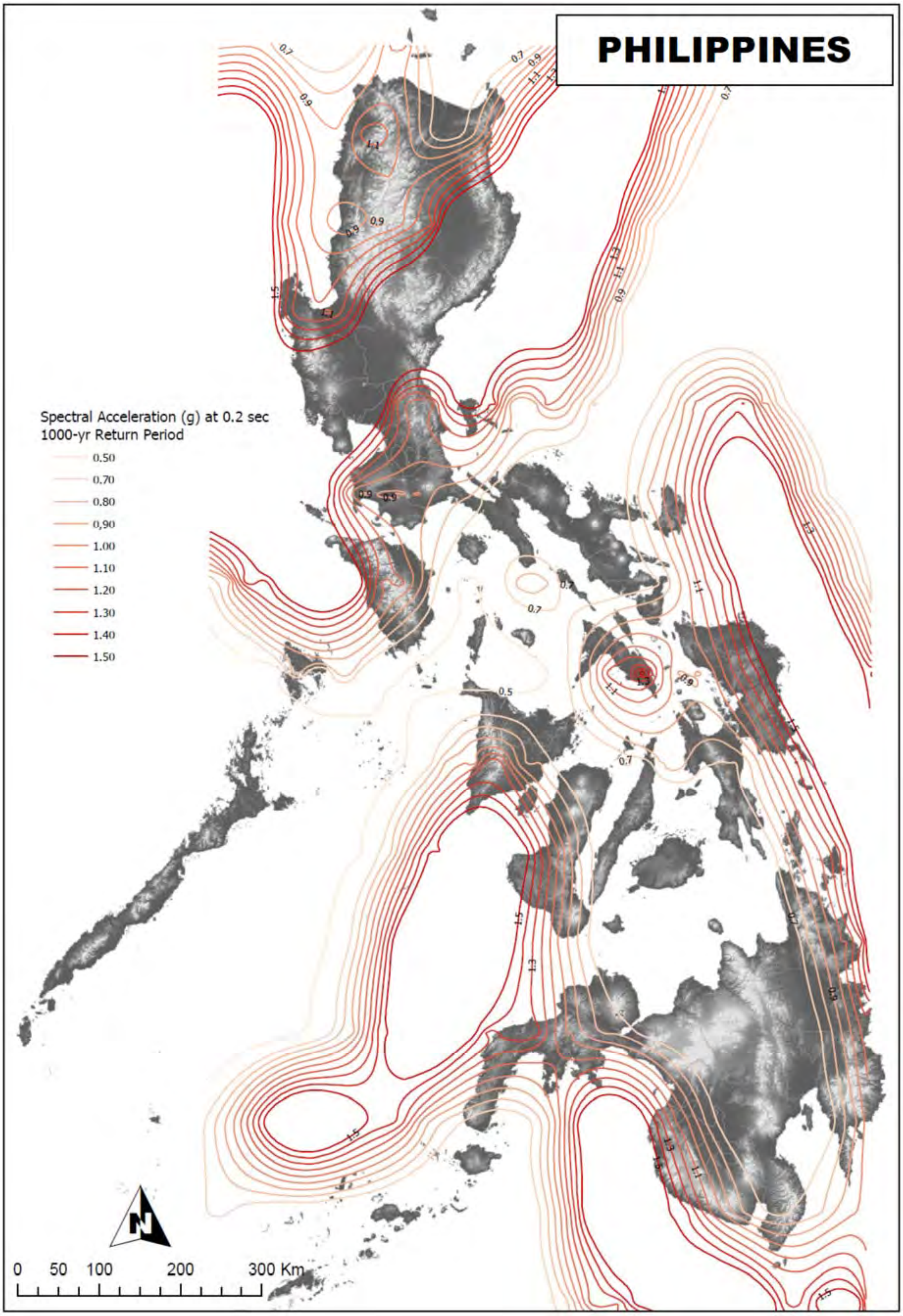


Figure 3.4.1-5 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_s) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

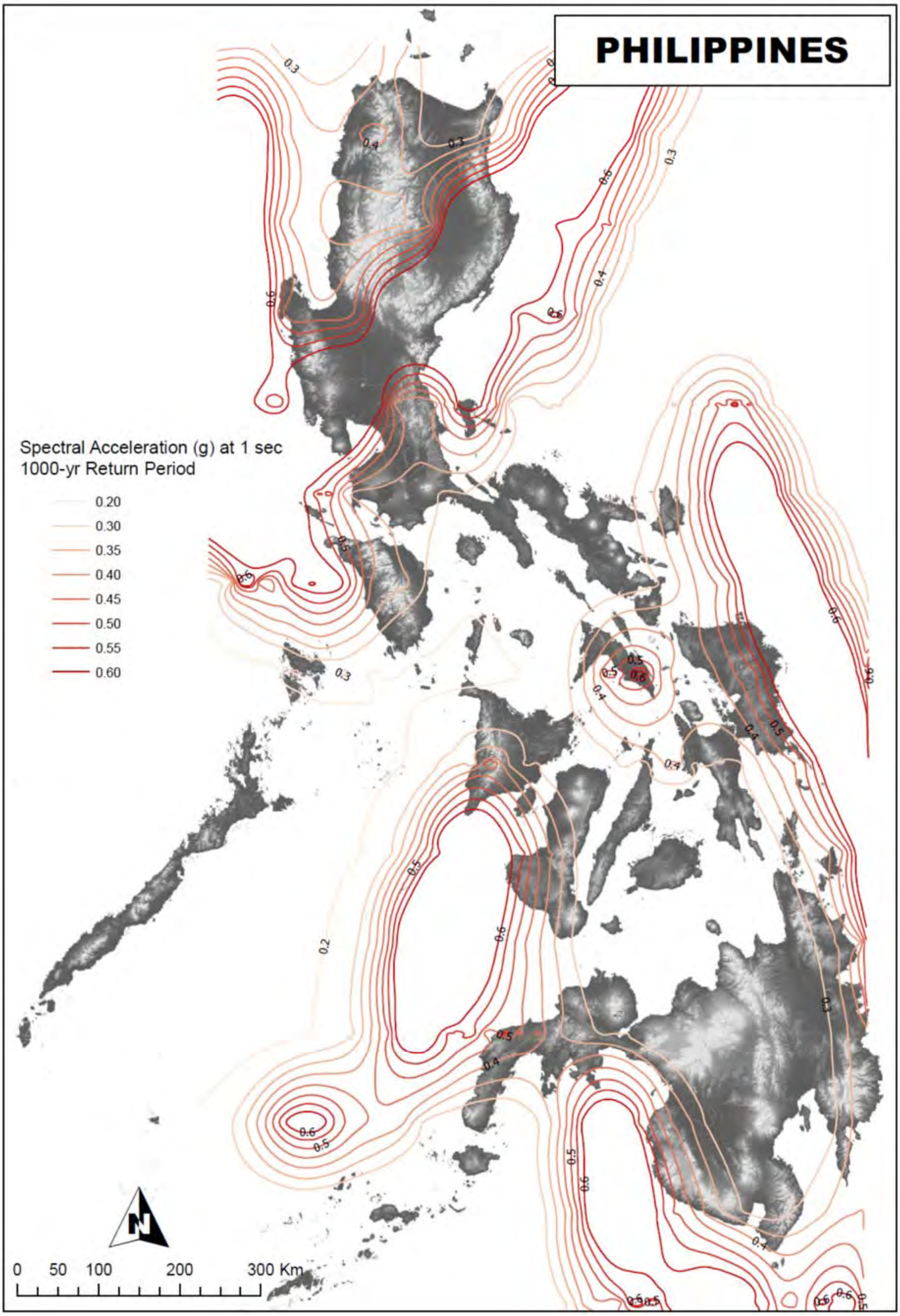


Figure 3.4.1-6 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

Commentary C3.4.1

(1) Values for the coefficients *PGA*, S_S and S_1 are expressed as ratios of gravitation acceleration (g) in Figures 3.4.1-1 to 3.4.1-3 for the Level 1 Earthquake Ground Motion and Figures 3.4.1-4 to 3.4.1-6 for Level 2 Earthquake Ground Motion for the entire Philippine archipelago. However, regional level acceleration coefficient maps are also provided in Appendix 3A and 3B for clearer identification of the location for a concerned site belonging to a particular region.

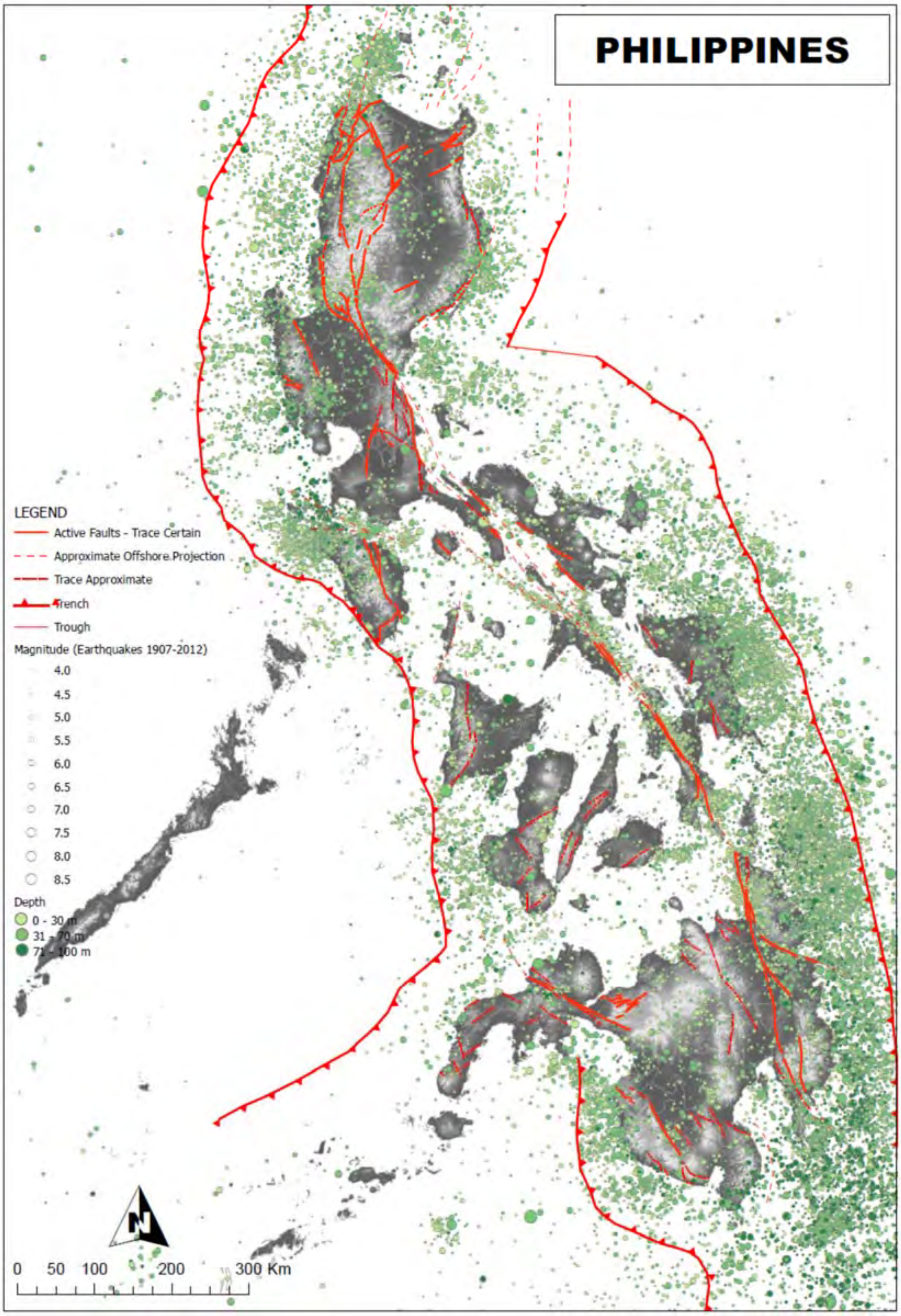


Figure C3.4.1-1 Philippine Earthquake Events from 1907 to 2012 with Magnitude > 4

The acceleration coefficient contour maps provided in these Specifications are derived based on the catalogue of earthquake events recorded in different parts of the Philippines, as shown in Figure C3.4.1-1. The probabilistic seismic hazard analysis (*PSHA*) is applied to the earthquake catalogue to determine the earthquake acceleration coefficients at the basement rock equivalent to the AASHTO Type B soil class. The AASHTO Site Class B is defined as rock with an average shear wave velocity in the upper 100ft (30.5m) of the soil profile in the range of 2,500 ft/s < $\tilde{v}_s$ < 5,000 ft/s (760 m/s < $\tilde{v}_s$ < 1,520 m/s).

Figures 3.4.1-1 to 3.4.1-3 for the Level 1 Earthquake Ground Motion have 100-year return periods or a probability of exceedance corresponding to 53% in 75 years. On the other hand, Figures 3.4.1-4 to 3.4.1-6 for Level 2 Earthquake Ground Motion have 1,000 year return periods or a probability of exceedance corresponding to 7% in 75 years.

The acceleration coefficients are based on a uniform risk model of seismic hazard. The probability that a coefficient will not be exceeded at a given location during a 75-year period is estimated to be about 93 percent for the Level 2 earthquake ground motion, i.e., a seven percent probability of exceedance. The use of a 75-year interval to characterize this probability is an arbitrary convenience and does not imply that all bridges are thought to have a useful life of 75 years. It can be shown that an event with this probability of exceedance has a return period of about 1,000 years and is called the design earthquake for Level 2 Earthquake Ground Motion. Larger earthquakes than that implied by the above set of coefficients have a finite probability of occurrence throughout the Philippines.

(2) When bridges are identified to be located between two contour lines in the acceleration coefficient maps provided in these Specifications, the value of the contour line with a higher acceleration coefficient shall be used as the design parameter values. This is to cover for the unknown factors and uncertainties due to limited data and knowledge of the site characteristics when applying *PSHA*.

3.4.2 Site Specific Procedure

(1) A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by DPWH as given in Article 3.4 and may be performed for any site. The objective of the site-specific probabilistic ground-motion analysis should be to generate a uniform-hazard acceleration response spectrum considering a seven percent (7%) probability of exceedance in 75 years (1,000-year return period for Level 2 earthquake ground motion) and fifty three percent (53%) probability of exceedance in 75 years (100-year return period for Level 1 earthquake ground motion) for spectral values over the entire period range of interest. This analysis should involve establishing:

- the contributing seismic sources;
- an upper-bound earthquake magnitude for each source zone;
- median attenuation relations for acceleration response spectral values and their associated standard deviations;
- a magnitude-recurrence relation for each source zone; and
- a fault-rupture-length relation for each contributing fault.

(2) Uncertainties in source modeling and parameter values shall be taken into consideration. Detailed documentation of ground-motion analysis is required and shall be peer reviewed.

(3) For sites located in close proximity to a known active surface or a shallow fault, as depicted in the PHIVOLCS Active Fault Map, studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response.

(4) Where analyses to determine site soil response effects are required, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses.

(5) A deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is no less than two-thirds of the probabilistic spectrum in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period. Where use of a deterministic spectrum is appropriate, the spectrum shall be either:

- the envelope of a median spectra calculated for characteristic maximum magnitude earthquakes on known active faults; or
- a deterministic spectra may be defined for each fault, and, in the absence of a clearly controlling spectra, each spectrum should be used.

(6) Where response spectra are determined from a site specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure of Article 3.4.1 in the region of $0.5T_F$ to $2T_F$ of the spectrum where T_F is the bridge fundamental period.

Commentary C3.4.2

(1) The intent in conducting a site-specific probabilistic ground motion study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from the generalized acceleration coefficient maps provided in these Specifications and the procedure of Article 3.4.1. Accordingly, such studies should be comprehensive and incorporate current scientific interpretations at a regional scale.

(2) Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground-motion attenuation, it is important to incorporate these uncertainties formally in a site-specific probabilistic analysis. Examples of these uncertainties include seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground-motion attenuation relationship.

(3) Near-fault effects on horizontal response spectra include:

- Higher ground motions due to the proximity of the active fault;
- Directivity effects that increase ground motions for periods greater than 0.5 s if the fault rupture propagates toward the site; and
- Directionality effects that increase ground motions for periods greater than 0.5 s in the direction normal (perpendicular) to the strike of the fault.

If the active fault is included and appropriately modeled in the development of national ground motion maps, then the first effect above is already included in the national ground motion maps. The second and third effects are not included in the national maps. These effects are significant only for periods longer than 0.5s and normally would be evaluated only for essential or critical bridges having natural periods of vibration longer than 0.5s. Further discussions of the second and third effects are contained in Somerville (1997) and Somerville et al. (1997).

The fault-normal component of near-field ($D < 10$km (6 mi.)) motion may contain relatively long-duration velocity pulses which can cause severe nonlinear structural response, predictable only through nonlinear time-history analyses. For this case the recorded near-field horizontal components of motion need to be transformed into principal components before modifying them to be response-spectrum-compatible.

The ratio of vertical-to-horizontal ground motions increases for short-period motions in the near-fault environment.

3.5 SITE EFFECTS

Ground Types (Site Classes) and Site Factors specified herein shall be used in the General Procedure for characterizing the seismic hazard specified in Article 3.6.

Commentary C3.5

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify ground motions in the underlying rock, sometimes by factors of two or more. The extent of this amplification is dependent on the profile of soil types at the site and the intensity of shaking in the rock below. Sites are classified by type and profile for the purpose of defining the overall seismic hazard, which is quantified as the product of the soil amplification and the intensity of shaking in the underlying rock.

Owing to the lack of data on the soil characteristics in the Philippines, previous design codes for bridges rely on the soil classification provided by AASHTO (Soil Profile Types I-IV, *AASHTO 2007*). However, such soil classification focusing on rock types predominant in the United States does not necessarily reflect the local ground conditions in the Philippines. However, the regional geologic setting classifies the Philippines and Japan into a kind of volcanic arc islands, and relatively geologically unstable land with similar tectonic setting. On the contrary, the United States, being one of the oldest continents on earth, is a continental crust and basically a stable land with predominant rock base. In this case, AASHTO has to cover a wide range of rock types compared to the Philippines and Japan with relatively younger geologic ages of rocks. Figure C3.5-1 illustrates the geological similarities between the Philippines, Japan and the United States.

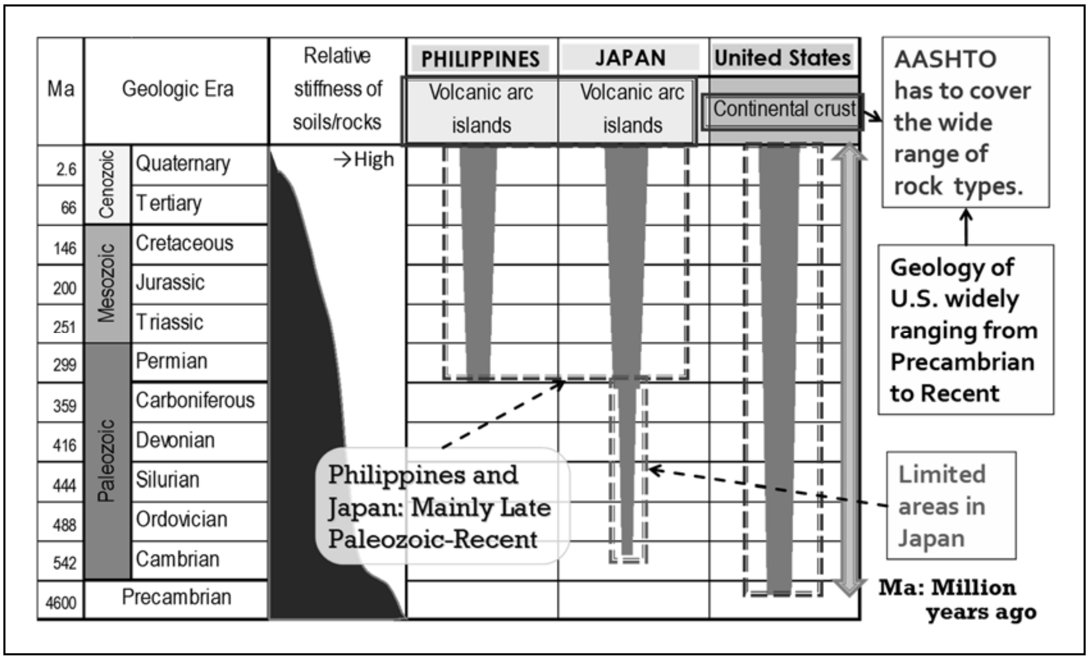


Figure C3.5-1 Geological Similarities between the Philippines, Japan and the United States

Until 2007, *AASHTO LRFD Bridge Design Specifications* provides a qualitative classification of soil profiles into Types I to IV which varies from rock, to stiff clays, to stable deposits and ground with 10m or more of soft to medium-stiff clays and soft clays or silts. However, the later versions of AASHTO LRFD gives a more definitive and quantitative classification of sites with class definitions from A through F, based on the ground stiffness determined by the shear wave velocity in the upper 30m of soil. However, emphasis is placed on rock sites which need a shear wave velocity measurement on site to determine the classification for rigid profile site class.

On the other hand, the Japan Road Association (JRA) provides a soil profile type classification system which is quantitative and has simple procedure. Three soil profile or ground types (Type I-III) are proposed to cover all kinds of ground conditions. Although the ground types are functions of the ground shear wave velocities, a simplified relation to determine the shear wave velocities using the SPT blow counts (N-values) is provided. This enables determination of the soil profile or ground type without undertaking in-situ shear wave measurement tests.

Table C3.5-1 compares the different soil profile types between AASHTO and JRA.

Table C3.5-1 Comparison of Soil Profile Types between AASHTO and JRA

AASHTO LRFD 2007	AASHTO LRFD 2012	JRA
Type I: Rock; stiff soils where soil depth is less than 60m; soil types overlying rocks are stable deposits of sand, gravels or stiff clays	Site Class A: Hard rock ($\tilde{v}_s$ > 5,000 ft/s) ($\tilde{v}_s$ > 1,520 m/s)	
	Site Class B: Rock (2,500 ft/s < $\tilde{v}_s$ < 5,000 ft/s) (760 m/s < $\tilde{v}_s$ < 1,520 m/s)	
	Site Class C: Very dense soil and soil rock (1,200 ft/s < $\tilde{v}_s$ < 2,500 ft/s) (365 m/s < $\tilde{v}_s$ < 760 m/s)	Type I: Good diluvial ground and rock
Type II: Stiff cohesive or deep cohesionless soils where the soil depth exceeds 60m with stable deposits of sands, gravels or stiff clays	Site Class D: Stiff soil (600 ft/s < $\tilde{v}_s$ < 1,200 ft/s) (180 m/s < $\tilde{v}_s$ < 365 m/s)	Type II: Diluvial and alluvial ground not belonging to either Type III or Type I ground
Type III: Soft to medium-stiff clays and sands with 9m or more of soft to medium stiff clays	Site Class E: Soil profile with more than 3m of soft clay ($\tilde{v}_s$ < 600 ft/s) ($\tilde{v}_s$ < 180 m/s)	Type III: Soft ground and alluvial ground
Type IV: Soft clay of silts greater than 12m in depth. ($\tilde{v}_s$ < 152 m/s)	Site Class F: Peats or highly organic clays; very high plasticity clays; very thick soft/medium stiff clays	

where $\tilde{v}_s$: average shear wave velocity for the upper 30.5m of the soil profile

These Specifications adopts the JRA ground type classification for the site class.

3.5.1 Ground Types Definitions (Site Class Definitions)

Ground types for seismic design shall be classified, in principle, in accordance with the types defined in Table 3.5.1-1, in accordance with the ground characteristic value T_G defined by Equation 3.5.1-1. When the ground surface lies on the same level as the surface of a base ground surface for seismic design, the ground type shall be Type I.

$$T_G = 4\sum_{i=1}^{n} \frac{H_i}{V_{si}} \quad \text{.......} \quad (3.5.1\text{-}1)$$

where:

T_G : Characteristic value of ground (s)

H_i : Thickness of the *i-th* soil layer (m)

V_{si} : Average shear elastic wave velocity of the i-th soil layer (m/s)

i : Number of the *i-th* soil layer from the ground surface when the ground is classified into n layers from the ground surface to the surface of the base ground surface for seismic design

Table 3.5.1-1 Ground Types (Site Class) for Seismic Design

Ground Type*	Soil Profile Description	Characteristic Value of Ground, T_G (s)
Type I	Hard (Good diluvial ground and rock)	$T_G < 0.2$
Type II	Medium (Diluvial and alluvial ground not belonging to Types I and III)	$0.2 \le T_G < 0.6$
Type III	Soft (Soft ground and alluvial ground)	$0.6 \le T_G$

** The Ground Type shall be determined quantitatively based on the Characteristic Value of Ground (T_G).*

Commentary C3.5.1

When the ground type for seismic design is determined according to the design earthquake ground motions as specified in Article 3.6, the effects of ground conditions shall be considered as described above. Generally, Type I ground includes good diluvial ground and rock, Type III ground includes soft ground and alluvial ground, and Type II ground denotes diluvial and alluvial ground not belonging to either Type III or Type I ground. The alluvial ground includes new sedimentary layers formed by landslides, surface soil, reclamation soil, soft soil or weak soil layers while compacted sand layer, gravel layer and boulder layer of alluvium origin may be treated as diluvial ground.

Ground types for seismic design shall be classified, in principle, into those types defined in Table 3.5.1-1, in accordance with the ground characteristic value T_G calculated from Equation 3.5.1-1. T_G, which originally refers to the fundamental natural period of the surface ground layer in the small strain amplitude region, is called the characteristic value of the ground. V_{si} in Equation 3.5.1-1, which is normally measured by elastic wave propagation method or PS logging, can be estimated using the N-value in case no measured value is available. To estimate V_{si}, the average N-values for each soil layer is used to simplify the calculation process in the following equations:

For cohesive soil layer,

$$V_{si} = 100N_i^{1/3}(1 \le N_i \le 25)$$

For sandy/cohesionless soil layer,

$$V_{si} = 80N_i^{1/3}(1 \le N_i \le 50) \quad \text{(C3.5.1-1)}$$

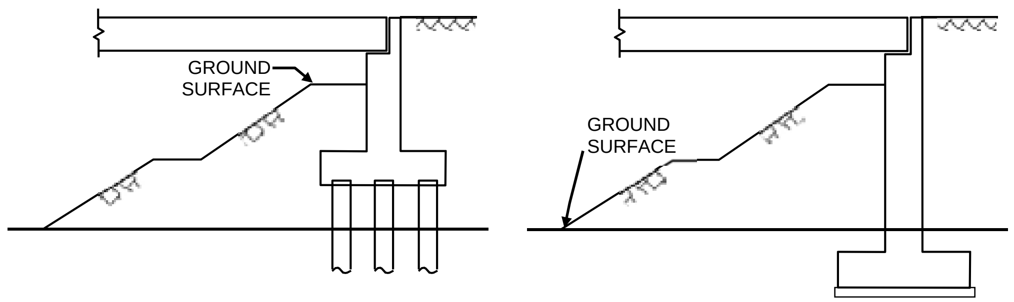
where:

N_i : Average *N-value* of the *i-th* soil layer obtained from SPT

Equation C3.5.1-1 is an approximation derived from experimental results in the range of N=1 to 25 for cohesive soil layers and in the range of N=1 to 50 for cohesionless/sandy layers. When the N-value is 0, the value of V_{si} can be taken as 50 m/s.

The surface of the base ground for seismic design represents the upper surface of a fully hard ground layer that exists over a wide area in the construction site, and is normally situated below the surface soil layer shaking with the ground motion during an earthquake. In which case, the upper surface of a fully hard ground layer may be the upper surface of a highly rigid soil layer with an elastic shear wave velocity of more than 300 m/s (for an N-value of 25 in the cohesive soil layer and an N-value of 50 in the sandy soil layer, as defined in Equation C3.5.1-1).

When the ground surface is not flat such as a levee body or fill, or the footing is constructed in the levee body as shown in Figure C3.5.1-1(a), the levee top should be regarded as the ground surface to obtain the characteristic value of ground for Equation 3.5.1-1 since the vibration of the substructure may be affected by the vibration of the levee body. However, when the footing is constructed in the ground below the levee, as shown in Figure C3.5.1-1(b), the average ground surface around it should be regarded as the ground surface in calculating the characteristic value of the ground.



(a) When the footing is constructed within the levee/high embankment body

(b) When the footing is constructed in the ground under the levee/high embankment body

Figure C3.5.1-1 Determining the Ground Surface of Levee Body and Fill

According to the results of calculations for different ground cases, it is clear that T_G has a correlation with the alluvial layer thickness H_A and the diluvial layer thickness H_D, for which the ground can be approximately classified into the types as shown in Figure C3.5.1-2. In this case, when it is difficult to obtain T_G from

Equation C3.5.1-1 as in the case when the ground surface cannot be identified even by a standard penetration test at a very deep layer, the ground type can be classified as shown in Figure C3.5.1-2.

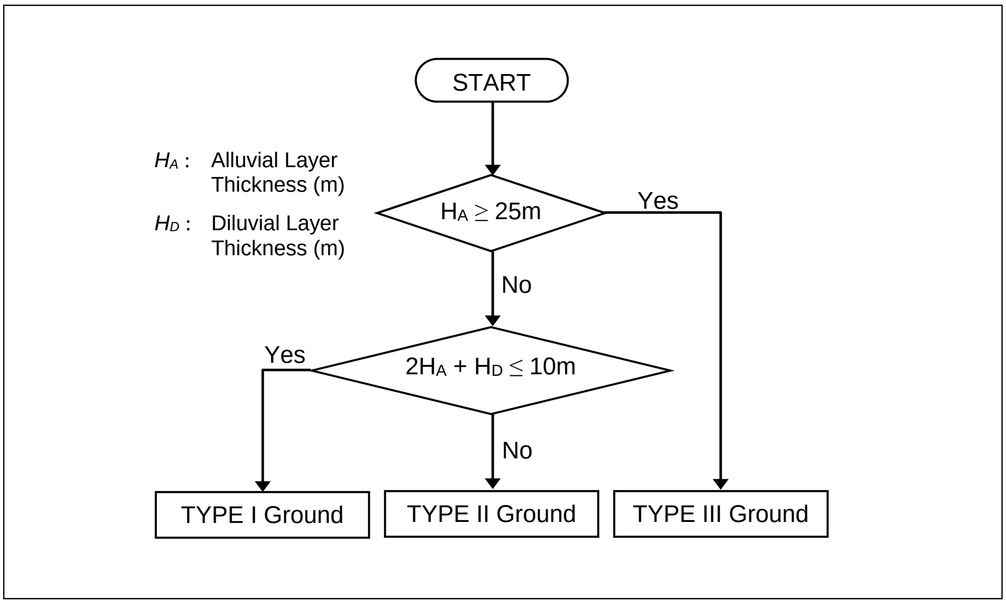


Figure C3.5.1-2 Ground Types Determined by Alluvial Layer Thickness H_A and Diluvial Layer Thickness H_D

3.5.2 Ground Surface in Seismic Design

The ground surface to be considered in seismic design shall generally be the ground surface during the normal design condition (limit states other than Extreme Event I under earthquake loading). However, for sites of unstable soil layers whose seismic subgrade reactions cannot be anticipated, the ground surface to be considered in seismic design shall be assumed approximately by taking the effects of unstable soil layers into account.

Commentary C3.5.2

The ground surface to be considered in seismic design refers to the ground surface in which the design earthquake ground motion/forces specified in Article 3.6 is applied, with the assumption that the seismic forces acts only on the structures above it and excluding the structures existing below it.

The ground surface to be considered in seismic design is defined as the ground surface in the normal design condition (limit states other than Extreme Event I under earthquake loading), as shown in Figures C3.5.2-1 and C3.5.2-2. However, for sites of unstable soil layers whose seismic subgrade reactions cannot be anticipated, such as an extremely soft layer or liquefiable sandy soil layer, the bottom face of the layer should be taken as the ground surface to be considered in seismic design to ensure safety design. This is because the quality of the ground of about 10m under the actual ground may largely affect the lateral resistance of the foundation and due to the uncertainty in the dynamic relation between the ground and the structure during an earthquake.

Basically, seismic design can be carried-out by considering the resistance of the ground around the footing and taking the upper part of the footing as the ground surface in the normal design condition, provided that the backfill around the footing is sufficiently done so that the ground surface is kept stable for a long period of time. However, in case that the weight of the footing may largely affect the whole foundation like pile foundation, it is recommended that the effects of the inertia force of the footing is taken into account in seismic design of foundation even if the ground surface in the normal design is just above the footing.

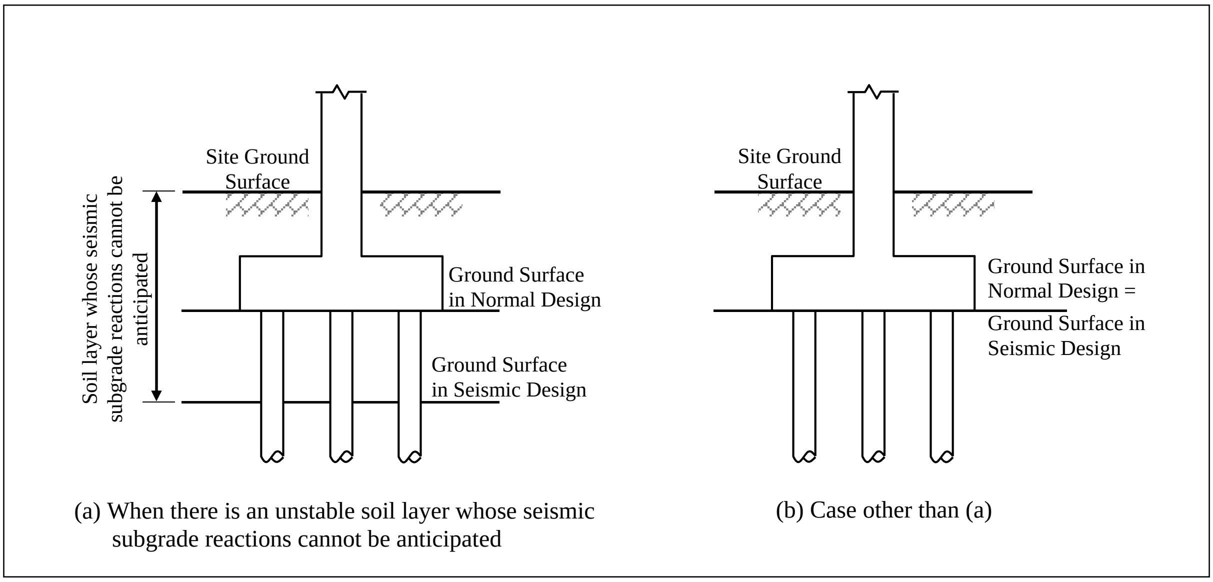


Figure C3.5.2-1 Ground Surface of Pier in Seismic Design

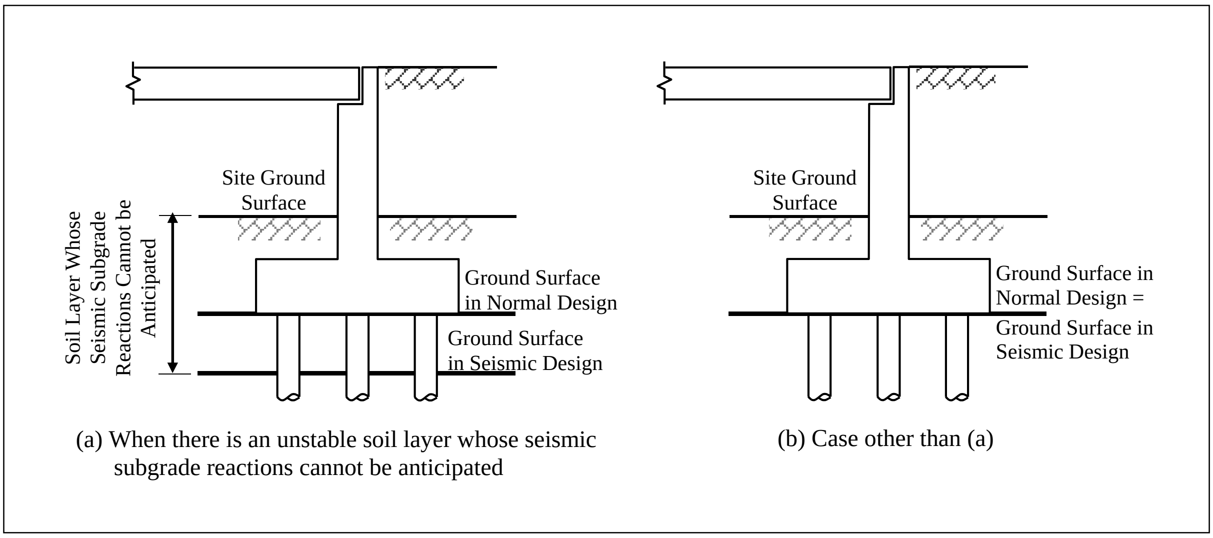


Figure C3.5.2-2 Ground Surface of an Abutment in Seismic Design

On sites where the soil layer strata alternates whose seismic subgrade reactions cannot be anticipated, the ground surface to be considered in seismic design should be the top face of the sandy soil layer with thickness of at least 3m and with none-zero subgrade reactions (see Figure C3.5.2-3). This is based on past earthquakes and the fact that the influence of a soil layer of zero subgrade reaction on a soil layer of none-zero subgrade reaction above it cannot be evaluated quantitatively.

For the case of an abutment, the ground surface to be considered in seismic design at different depths may be set at either the rear or the front of the abutment. However, for safety design, it is generally set in front of the abutment. The seismic earth pressure acting on an abutment from its back side should be assumed to act above the bottom face of the footing at the rear of the abutment, irrespective of whether the ground surface to be considered in seismic design is at the bottom face of the footing at the rear of the abutment or further below it. In design, it is recommended that the lateral resistance of the ground existing above the ground surface be disregarded for seismic design in front of the abutment.

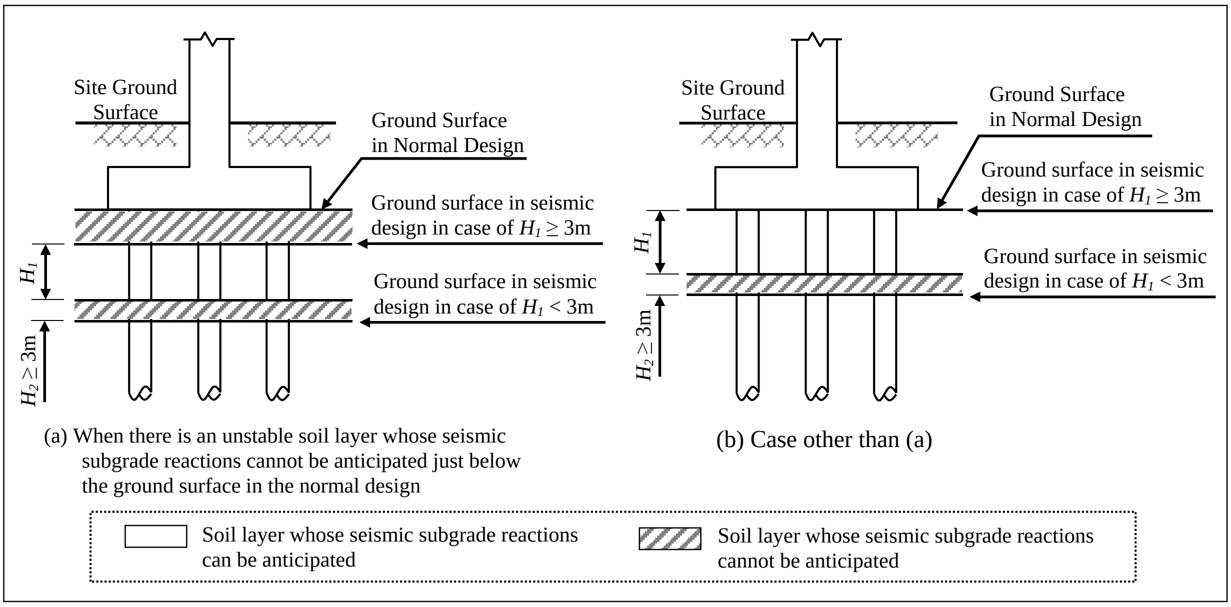


Figure C3.5.2-3 Ground Surface to be Considered in Seismic Design When There Is An Intermediate Soil Layer Whose Seismic Subgrade Reactions Can be Anticipated

3.5.3 Site Factors

The Site Factors F_{pga}, F_a and F_v specified in Tables 3.5.3-1, 3.5.3-2, and 3.5.3-3 shall be used in the zero-period, short-period range, and long-period range, respectively for the elastic seismic response coefficient in the design response spectrum of Article 3.6 of this Section. These factors shall be determined using the Ground Types (Site Class) given in Table 3.5.1-1 and the mapped values of the coefficients PGA, S_S, and S_1 in Figures 3.4.1-1 to 3.4.1-6 and Appendix 3A and 3B.

Table 3.5.3-1 Values of Site Factor, F_{pga} at Zero-Period on Acceleration Spectrum

Ground Type (Site Class)	Peak Ground Acceleration Coefficient (PGA)[1]					
	$PGA \leq 0.10$	$PGA = 0.20$	$PGA = 0.30$	$PGA = 0.40$	$PGA = 0.50$	$PGA \geq 0.80$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note:
[1] Use straight-line interpolation for intermediate values of *PGA*.

Table 3.5.3-2 Values of Site Factor, F_a, for Short-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 2.0$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note:
[1] Use straight-line interpolation for intermediate values of S_S.

Table 3.5.3-3 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.80$
I	1.7	1.6	1.5	1.4	1.4	1.4
II	2.4	2.0	1.8	1.6	1.5	1.5
III	3.5	3.2	2.8	2.4	2.4	2.0

Note:
[1] Use straight-line interpolation for intermediate values of S_1.

Commentary C3.5.3

The peak ground acceleration coefficient maps and the response spectral acceleration coefficient maps shown in Figures 3.4.1-1 to 3.4.1-6 are generated using the equivalent Class B soil profile (soft rock) of the *AASHTO LRFD Bridge Design Specifications (2012)* as the reference site category for basement rock. Site Class B rock is therefore the site condition for which the site factor is 1.0. In this regard, Ground Types I – III of these Specifications shall have separate sets of site factors for zero-period (F_{pga}), short-period range (F_a), and long-period range (F_v) as indicated in Tables 3.5.3-1 to 3.5.3-3. These site factors generally increase as the soil profile becomes softer (in going from Ground Type I-III). The factors also decrease as the earthquake ground motion level increases, due to the strongly nonlinear behavior of the soil. For a given Ground Type I, II or III (Site Class C, D, or E in AASHTO), these nonlinear site factors increase more in the areas having lower rock ground motions than in the areas having higher rock ground motion.

The basis of the site factors (F_{pga}, F_a, F_v) above are taken from the equivalent site factors of the site class types based on the *AASHTO LRFD Bridge Design Specifications (2012)* and the *JRA* design response spectra for different ground types. The AASHTO Specifications uses Site Class B (soft rock) as the reference site category for the USGS and NEHRP MCE ground shaking maps and applies the site factor of 1.0 for Site class B rock. Since Ground Types I – III of these Specifications have similar characteristics with AASHTO Site Class C – E, AASHTO site factors are used as a reference and compared to the site factors of JRA design response spectra. The JRA design response spectra is generated based on the actual records of ground motion in Japan corresponding to the soil profile or ground types Type I to III. Moreover, since AASHTO site factors are given up to PGA of 0.50 only, an additional column for site factors for PGA of 0.80 is added based on the ground acceleration record experienced in Japan.

Tables C3.5.3-1 to C3.5.3-3 are the AASHTO site factors for Sites Classes C, D and E and are shown here for reference.

Table C3.5.3-1 Values of Site Factor, F_{pga} at Zero-Period on Acceleration Spectrum (AASHTO)

Site Class	Peak Ground Acceleration Coefficient (PGA)				
	$PGA < 0.10$	$PGA = 0.20$	$PGA = 0.30$	$PGA = 0.40$	$PGA > 0.50$
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

Table C3.5.3-2 Values of Site Factor, F_a, for Short-Period Range on Acceleration Spectrum (AASHTO)

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)				
	$S_S < 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S > 1.25$
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

Table C3.5.3-3 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum (AASHTO)

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)				
	$S_1 < 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 > 0.50$
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

Using the JRA design response spectra, the site factors F_{pga}, F_a, F_v are calculated at a certain range of PGA shown in Tables C3.5.3-4 and C3.5.3-5. These calculated values are used as reference for the site factors specified in these Specifications.

Table C3.5.3-4 Calculated Site Factors, F_{pga} and F_a based on JRA Design Response Spectra

A_s and S_{DS} Corresponding to PGA															
PGA (Soil Type B)		PGA=0.10	S_{DS}/As	PGA =0.20	S_{DS}/As	PGA =0.30	S_{DS}/As	PGA =0.40	S_{DS}/As	PGA =0.50	S_{DS}/As	PGA =0.60	S_{DS}/As	PGA =0.80	S_{DS}/As
Reference (Soil Type B)	**Fpga(=Fa)**	1.00		1.00		1.00		1.00		1.00		1.00		1.00	
	As(Fpga*PGA)	0.10	2.50	0.20	2.50	0.30	2.50	0.40	2.50	0.50	2.50	0.60	2.50	0.80	2.50
	S_{DS}(Fa*Ss)	0.25		0.50		0.75		1.00		1.25		1.50		2.00	
Soil Type I (C)	**Fpga(=Fa)**	1.20		1.20		1.10		1.00		1.00		1.00		1.00	
	As(Fpga*PGA)	0.12	2.50	0.24	2.50	0.33	2.50	0.40	2.50	0.50	2.50	0.60	2.50	0.80	2.50
	S_{DS}(Fa*Ss)	0.30		0.60		0.83		1.00		1.25		1.50		2.00	
Soil Type II (D)	**Fpga(=Fa)**	1.60		1.40		1.20		1.00		0.90		0.89		0.88	
	As(Fpga*PGA)	0.16	2.50	0.28	2.50	0.36	2.50	0.40	2.50	0.45	2.51	0.54	2.50	0.70	2.50
	S_{DS}(Fa*Ss)	0.40		0.70		0.90		1.00		1.13		1.35		1.75	
Type III (E, F)	**Fpga(=Fa)**	2.50		1.70		1.20		0.90		0.80		0.77		0.75	
	As(Fpga*PGA)	0.25	2.50	0.34	2.50	0.36	2.50	0.36	2.50	0.40	2.50	0.47	2.51	0.60	2.50
	S_{DS}(Fa*Ss)	0.63		0.85		0.90		0.90		1.00		1.18		1.50	

Site specific response spectra procedure was conducted under this project to determine the design acceleration response spectra for the basic design of seven (7) bridges indicated in Table C3.5.3-6 below. Geotechnical investigation with downhole shear wave velocity test was conducted for these bridges to determine the soil profile, geotechnical design parameters, potential for liquefaction and the corresponding ground types for seismic analysis. As indicated in table Table C3.5.3-6, most of the seven bridges are located in Ground Types II and III. Based on the calculated design acceleration response spectra, the site factors F_{pga},

F_a, F_v are calculated with the results shown in Table C3.5.3-6. The results indicate that the site factors values recommended under these Specifications are reasonable. However, more verification study based on actual experience of soil profile and earthquake ground motions are needed to fully localize the site factors indicated in these Specifications.

Table C3.5.3-5 Calculated Site Factors F_v based on JRA Design Response Spectra

Values of F_v (JRA)					
PGA (Soil Type B)		**S_1=0.50**	**$T_s = S_{D1}/S_{DS}$**	**S_1=0.80**	**$T_s = S_{D1}/S_{DS}$**
Reference (Soil Type B)	F_v	**1.00**		**1.00**	
	$S_{D1}(F_v*S_1)$	0.50	0.36	0.80	0.40
	S_{DS}	1.40		2.00	
Soil Type I (C)	F_v	**1.39**		**1.38**	
	$S_{D1}(F_v*S_1)$	0.70	0.50	1.10	0.55
	S_{DS}	1.40		2.00	
Soil Type II (D)	F_v	**1.50**		**1.50**	
	$S_{D1}(F_v*S_1)$	0.75	0.58	1.20	0.71
	S_{DS}	1.30		1.70	
Type III (E)	F_v	**2.40**		**1.88**	
	$S_{D1}(F_v*S_1)$	1.20	1.00	1.50	1.00
	S_{DS}	1.20		1.50	

Table C3.5.3-6 Site Factors F_v Calculated for 7 Bridges based on Site Specific Design Response Spectra

Characteristics of Acceleration Response Spectra for Target Seven (7) Bridges (1000-yr Return Period)													
Bridge Name	**Soil Type**	**F_{pga} & F_a**							**F_v**				**Remarks**
		$A_s = F_{pga}$ * PGA			**$S_{DS} = F_a * S_s$**			**S_{DS}/A_s**	**$S_{D1} = F_v * S_1$**			**$T_s = S_{D1}/S_{DS}$**	
		PGA	**F_{pga}**	**A_s**	**S_s**	**F_a**	**S_{DS}**		**S_1**	**F_v**	**S_{D1}**		
B-08 Lambingan Br.	II	0.46	0.83	0.38	1.06	0.87	0.92	2.42	0.40	1.28	0.51	0.55	T_G = 0.43s
	III	0.46	0.83	0.38	1.06	0.87	0.92	2.42	0.40	1.28	0.51	0.55	T_G = 0.75s
B-10 Guadalupe Br.	II	0.47	0.81	0.38	1.08	0.85	0.92	2.42	0.40	1.40	0.56	0.61	T_G = 0.32 ~ 0.49s
	III	0.47	0.81	0.38	1.08	0.86	0.93	2.45	0.40	1.28	0.51	0.55	T_G = 0.93s
C-07 1st Mandaue-Mactan Br.	II	0.25	1.04	0.26	0.56	1.14	0.64	2.46	0.21	1.62	0.34	0.53	T_G = 0.42s
	III	0.25	1.04	0.26	0.56	1.09	0.61	2.35	0.21	2.29	0.48	0.79	T_G = 1.20s
C-09 Palanit Br.	I	0.51	1.23	0.63	1.12	1.40	1.57	2.49	0.48	1.10	0.53	0.34	T_G = 0.19s
C-11 Mawo Br.	II	0.52	0.81	0.42	1.14	0.83	0.95	2.26	0.48	1.73	0.83	0.87	T_G = 0.46s
	III	0.52	0.73	0.38	1.14	0.72	0.82	2.16	0.48	1.92	0.92	1.12	T_G = 1.36s
C-14 Liloan Br.	II	0.36	1.00	0.36	0.81	1.10	0.89	2.47	0.33	1.61	0.53	0.60	T_G = 0.32s
C-15 Wawa Br.	II	0.45	0.84	0.38	1.00	0.88	0.88	2.32	0.38	1.68	0.64	0.73	T_G = 0.41s

3.6 SEISMIC HAZARD CHARACTERIZATION

3.6.1 Design Response Spectrum

The five-percent-damped-design response spectrum shall be taken as specified in Figure 3.6.1-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from Figures 3.4.1-1 to 3.4.1-6, scaled by the zero-, short-, and long-period site factors, F_{pga}, F_a, and F_v, respectively.

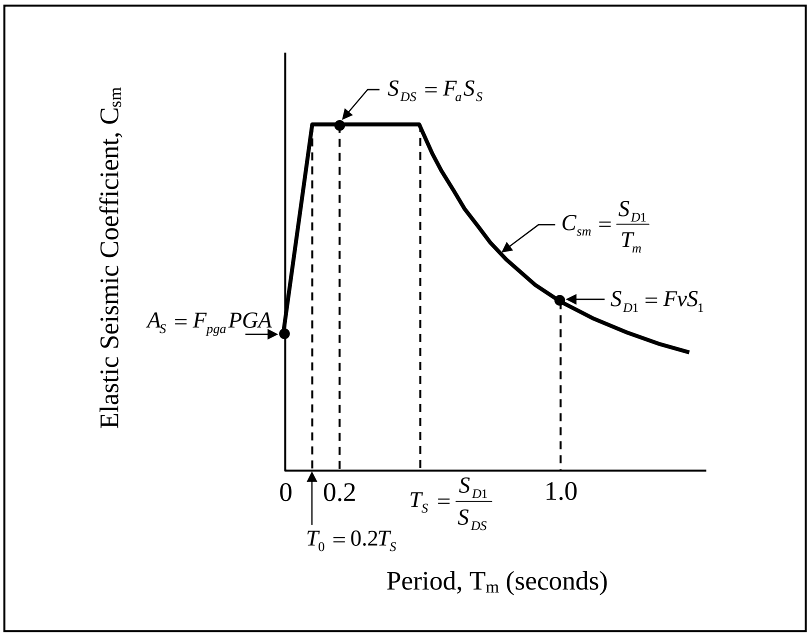


Figure 3.6.1-1 Design Response Spectrum

Commentary C3.6.1

The long-period portion of the response spectrum in Figure 3.6.1-1 is inversely proportional to the period, T. In the previous edition of the AASHTO Specifications, this portion of the spectrum was inversely proportional to $T^{2/3}$. The consequence of this change is that spectral accelerations at periods greater than 1.0 second are smaller than previously specified (for the same ground acceleration and soil type), and greater than previously specified for periods less than 1.0 s (but greater than T_s). This change is consistent with the observed characteristics of response spectra calculated from recorded ground motions. This revised shape is recommended in recent publications by NCHRP (2002, 2006), MCEER/ATC (2003), and FHWA (2006).

For periods exceeding about 3 seconds, it has been observed that in certain seismic environments spectral displacements tend to a constant value which implies that the acceleration spectrum becomes inversely proportional to T^2 at these periods. As a consequence, the spectrum in Figure 3.6.1-1 and Equation 3.6.2-5 may give conservative results for long period bridges (greater than about 3 seconds),

3.6.2 Elastic Seismic Response Coefficient

(1) For periods less than or equal to T_0, the elastic seismic coefficient for the *m*th mode of vibration, C_{sm}, shall be taken as:

$$C_{sm} = A_s + (S_{DS} - A_S)\,(T_m/T_0) \qquad (3.6.2\text{-}1)$$

in which:

$$A_S = F_{pga}\,PGA \qquad (3.6.2\text{-}2)$$

$$S_{DS} = F_a\,S_S \qquad (3.6.2\text{-}3)$$

where:

C_{sm} : elastic seismic response coefficient

A_S : effective peak ground acceleration coefficient

F_{pga} : site coefficient for peak ground acceleration specified in Article 3.5.3

PGA : peak ground acceleration coefficient on rock (equivalent to AASHTO Site Class B)

F_a : site coefficient for 0.2-sec period spectral acceleration specified in Article 3.5.3

S_S : horizontal response spectral acceleration coefficient at 0.2-sec period on rock (equivalent to AASHTO Site Class B)

T_m : period of vibration of *m*th mode, (s)

T_0 : reference period used to define spectral shape = $0.2T_s$ (s)

T_S : corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (s)

(2) For periods greater than or equal to T_0 and less than or equal to T_s, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{DS} \qquad (3.6.2\text{-}4)$$

(3) For periods greater than T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{D1}/T_m \qquad (3.6.2\text{-}5)$$

in which:

$$S_{D1} = F_v\,S_1 \qquad (3.6.2\text{-}6)$$

where:

F_v : site coefficient for 1.0-sec period spectral acceleration specified in Article 3.5.3.

S_1 : horizontal response spectral acceleration coefficient at 1.0 second period on rock (equivalent to AASHTO Site Class B)

Commentary C3.6.2

(1) An earthquake may excite several modes of vibration in a bridge and, therefore, the elastic response coefficient should be found for each relevant mode.

The discussion of the single-mode method in the commentary to Article 4.1.3.2 illustrates the relationship between period, C_{sm} and quasi-static seismic forces, $p_e(x)$. The structure is analyzed for these seismic forces in the single-mode method. In the multimode method, the structure is analyzed for several sets of seismic forces, each corresponding to the period and mode shape of one of the modes of vibration, and the results are combined using acceptable methods, such as the Complete Quadratic Combination (CQC) method as required in Article 4.1.3.3. C_{sm} applies to weight, not mass.

3.7 SEISMIC PERFORMANCE ZONES (SPZ)

Each bridge shall be assigned to one of the four Seismic Performance Zones in accordance with Table 3.7-1 using the value of S_{D1} (based on the 1-sec period design spectral acceleration for the design earthquake) given by Equation 3.6.2-6. The SPZ for the bridge shall be based on the Level 2 Earthquake Ground Motion as the design earthquake with 1,000-year return period.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with Seismic Performance Zone 4 (SPZ-4), regardless of the magnitude of S_{D1}.

Table 3.7-1 Seismic Performance Zones (SPZ)

Acceleration Coefficient, S_{D1}	Seismic Performance Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

Commentary C3.7

The seismic performance zones (SPZ) correspond to the seismic zones defined in Table 3.10.6-1 of the *AASHTO LRFD Bridge Design Specifications (2012)* and are synonymous to the seismic design categories (SDC) of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, which reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures. The requirements for the seismic performance zones in these Specifications shall comply with the requirements of the seismic zones in the *AASHTO LRFD Bridge Design Specifications (2012)*.

The seismic hazard level is defined as a function of the magnitude of the ground surface shaking as expressed by F_vS_1. However, other factors may affect the SPZ selected. For example, if the soil is liquefiable and lateral spreading or slope failure can occur, SPZ 4 should be selected. For assessment of existing structures, the Designer should also consider using SPZ 4 regardless of the magnitude of *As*, even when significant lateral soil movement is not expected, if the structure is particularly weak with regard to its stability to resist the forces and displacements that could be caused by the liquefaction.

3.8 RESPONSE MODIFICATION FACTORS

3.8.1 General

(1) To apply the response modification factors specified herein, the structural details shall satisfy the provisions of *Articles 5.10.2.2 (Concrete Structures, Seismic Hooks – Details of Reinforcement), 5.10.11 (Concrete Structures, Provisions for Seismic Design – Details of Reinforcement), and 5.13.4.6 (Concrete Structures, Seismic Requirements - Specific Members) of the AASHTO LRFD Bridge Design Specifications (2012 or later)*. The structural details shall likewise comply with the revised and updated DPWH Guide Specifications.

(2) Except as noted herein, seismic design force effects for substructures and the connections between parts of structures, listed in Table 3.8.1-2, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, *R*, as specified in Tables 3.8.1-1 and 3.8.1-2, respectively.

(3) As an alternative to the use of the R-factors, specified in Table 3.8.1-2 for connections, monolithic joints between structural members and/or structures, such as a column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent they connect as specified in Article 4.2.2.4.

(4) If an inelastic time history method of analysis is used, the response modification factor, *R*, shall be taken as 1.0 for all substructure and connections.

Table 3.8.1-1 Response Modification Factors – Substructures

Substructure	**Operational Category**		
	OC-I (Critical)	**OC-II (Essential)**	**OC-III (Others)**
Wall-type piers – larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 2.0 1.5	 3.0 2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents • Vertical piles only • With batter piles	 1.5 1.5	 3.5 2.0	 5.0 3.0
Multiple column bents	1.5	3.5	5.0

Table 3.8.1-2 Response Modification Factors – Connections

Connection	All Operational Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beams or superstructure	1.0
Columns or piers to foundations	1.0

Commentary C3.8.1

(1) These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate *R*-factor. In this case, the structure, particularly columns, should have enough ductility to be able to deform inelastically to the deformations caused by large earthquakes without loss of post-yield strength. The force-reduction factors are then specified to determine the inelastic deformation demands on the bridge members when the design earthquake occurs.

The *AASHTO Standard Specifications for Highway Bridges (WSD/LFD)*, 1992 edition, which is based on the ATC-6 Publication in 1981, specifies determination of the seismic design forces by dividing the elastic member demand forces obtained from the response spectrum analysis by an appropriate response modification factor, R (also referred to as the force-reduction factor) depending on the substructure type being considered. The R-factor, which is independent of the bridge classification, basically represents the column ductility demand with consideration of ductility capacity, overstrength and structural redundancy. These R-factors, is also used in the AASHTO 1996 edition for all bridge importance classification.

Moreover, with the release of the *AASHTO LRFD Bridge Design Specifications (1st Ed.)* in 1994, the column ductility is likewise accounted for by using the response modification factor R. The single values for the R-factors in the AASHTO 1996 edition was increased in the AASHTO LRFD 2012 edition (same as the 1994 LRFD edition) to three levels to account for the three bridge operational categories (“Critical, “Essential” and “Other”), as indicated in Table 3.8.1-1. However, the values of the response modification factors, R, for the 1996 WSD/LFD corresponds to the “Others” Operational Classification of the *2012 AASHTO LRFD Bridge Design Specifications* as indicated in Table 3.8.1-1. Note that the 1996 R-factors are specified both in the NSCP 2005 and the DPWH Guidelines (2004 Draft).

It should be noted that the R-factor method uses the equal displacement approximation which assumes that the maximum displacement of an elastic system is the same (or very close) to that of an inelastic system when subjected to the same design earthquake. For instance, based on the equal-displacement approximation the two structures having the same lateral stiffness, but with different yield strengths and subject to an elastic lateral force, will have the same maximum inelastic deformation. In this case, the ratio of the inelastic strength demand to the elastic strength demand denotes the force-reduction factor, R. Moreover, when considering the equal-displacement approximation assumption, the force reduction factors R1 and R2, will likewise represents the member ductility demands. However, the R-factor method assumes that the structural strength and stiffness are independent, that the ductility demand is the same for each type of structure and that the strength controls the damage to the structure. On the contrary, sound seismic design practice stipulates that the ductility capacity of structures should be greater than the seismic induced ductility demand. This leads to some problems attributed to the use of the R-factor design method. The drawback of the R-factor method is discussed in the paper “*J. Ger and F.Y. Cheng, ”Seismic Design Aids for Non-Linear Pushover Analysis of Reinforced Concrete and Steel Bridges”, CRC Press, 2012 (Taylor & Francis Group, LLC)”*.

(2) R-factors for connections are smaller than those for substructure members in order to preserve the integrity of the bridge under these extreme loads. For expansion joints within the superstructure and connections between the superstructure and abutment, the application of the R-factor results in force effect magnification. Connections that transfer forces from one part of a structure to another include, but are not limited to, fixed bearings, expansion bearings with either restrainers, shock transmission units (STUs), or dampers, and shear keys. For one-directional bearings, these R-factors are used in the restrained direction only. In general, forces determined on the basis of plastic hinging will be less than those given by using Table 3.8.1-2, resulting in a more economical design.

Table 3.8.1-1 shows the *R*-factors for the three operational categories with critical bridges having the lowest modification factors and thus the biggest earthquake demand forces. However, the previous version of AASHTO (which is applied to the design of DPWH bridges prior to these Specifications) classifies bridges under two importance categories – critical/essential and others. The *R*-factor values applied for critical/essential bridges in the previous AASHTO Specifications is equivalent to the *R*-factor values under OC-III (Others) category of Table 3.8.1-1. This implies that the present Specifications will have larger demand forces under earthquake loading. However, the present Specifications will adopt the localized seismic PGA and spectral acceleration coefficients developed considering the Philippine fault system and the plate boundaries as the source of ground excitation, contrary to the previous design requirements of DPWH bridges using 0.40 and 0.20 as the PGA coefficients. In this case, some areas in the Philippines may have higher earthquake demand forces but other areas may have lesser demand forces.

In view of the performance requirements specified in Table 3.2-1 and the large earthquake demand forces expected for critical bridges (OC-I), the DPWH classifies only major and important bridges under the OC-I category.

3.8.2 Application

(1) Seismic loads shall be assumed to act in any lateral direction.

(2) The appropriate R-factor shall be used for both orthogonal axes of the substructure.

(3) A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in *Section 5 Concrete Structures of the AASHTO LRFD Bridge Design Specifications (2012 or later)*, are satisfied.

Commentary C.3.8.2

(1) Usually the orthogonal axes will be the longitudinal and transverse axes of the bridge. In the case of a curved bridge, the longitudinal axis may be the chord joining the two abutments.

(2) Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.

3.9 REQUIREMENTS FOR TEMPORARY BRIDGES AND STAGE CONSTRUCTION

(1) Any bridge or partially constructed bridge that is expected to be temporary for more than 5 years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

(2) The seismic performance requirements stated in Article 3.3 shall apply to temporary bridges expected to carry traffic. It shall also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The elastic seismic response coefficient and the ground acceleration coefficient given in Article 3.6.2 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Response and acceleration coefficients for construction sites that are close to active faults shall be subject to special study. The response modification factors given in Article 3.8 may be

increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 3.8.1-2.

(3) The minimum support length provisions of Article 4.6 shall apply to all temporary bridges and staged construction.

Commentary C3.9

(1) The option to use a reduced response coefficient and a reduced ground acceleration coefficient reflects the limited exposure period for a temporary bridge.

APPENDIX 3A:

SPECTRAL ACCELERATION MAPS FOR LEVEL 1 EARTHQUAKE GROUND MOTION

(100-YEAR RETURN PERIOD)

Appendix 3A presents the spectral acceleration maps for Level 1 Earthquake Ground Motion (100-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)

(100-YEAR RETURN PERIOD)

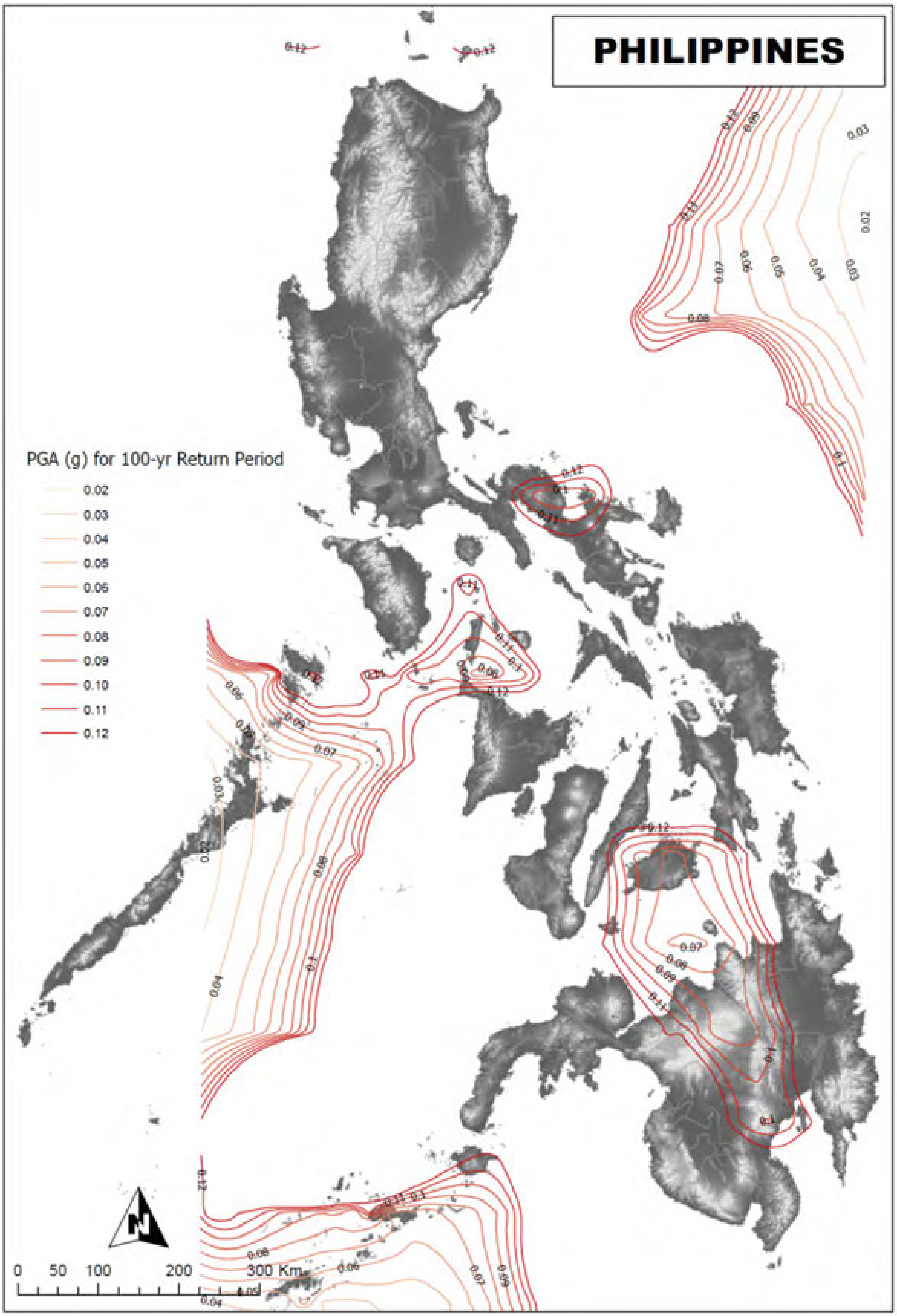


Figure 3A-1 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Entire Country)

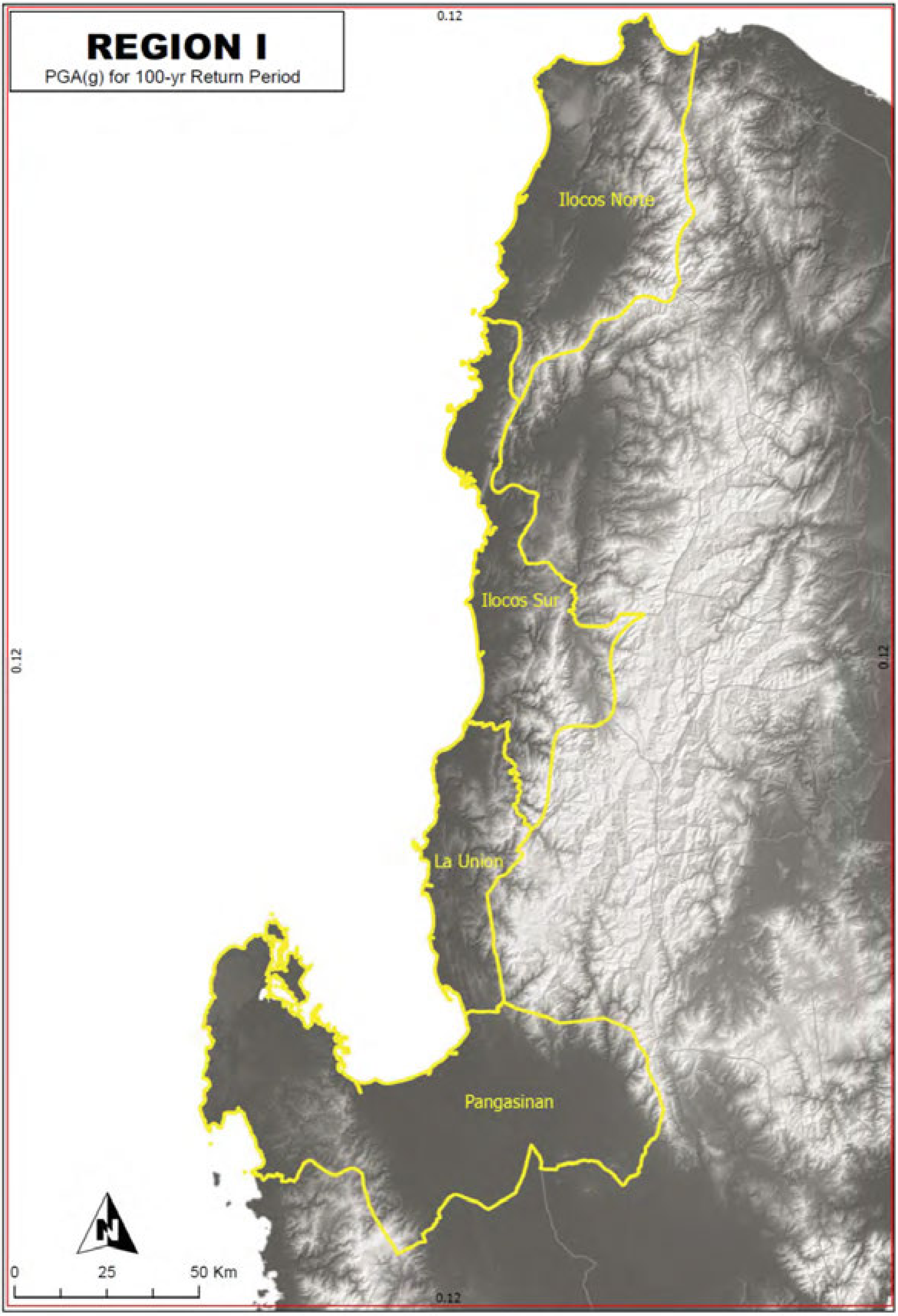


Figure 3A-2 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region I)

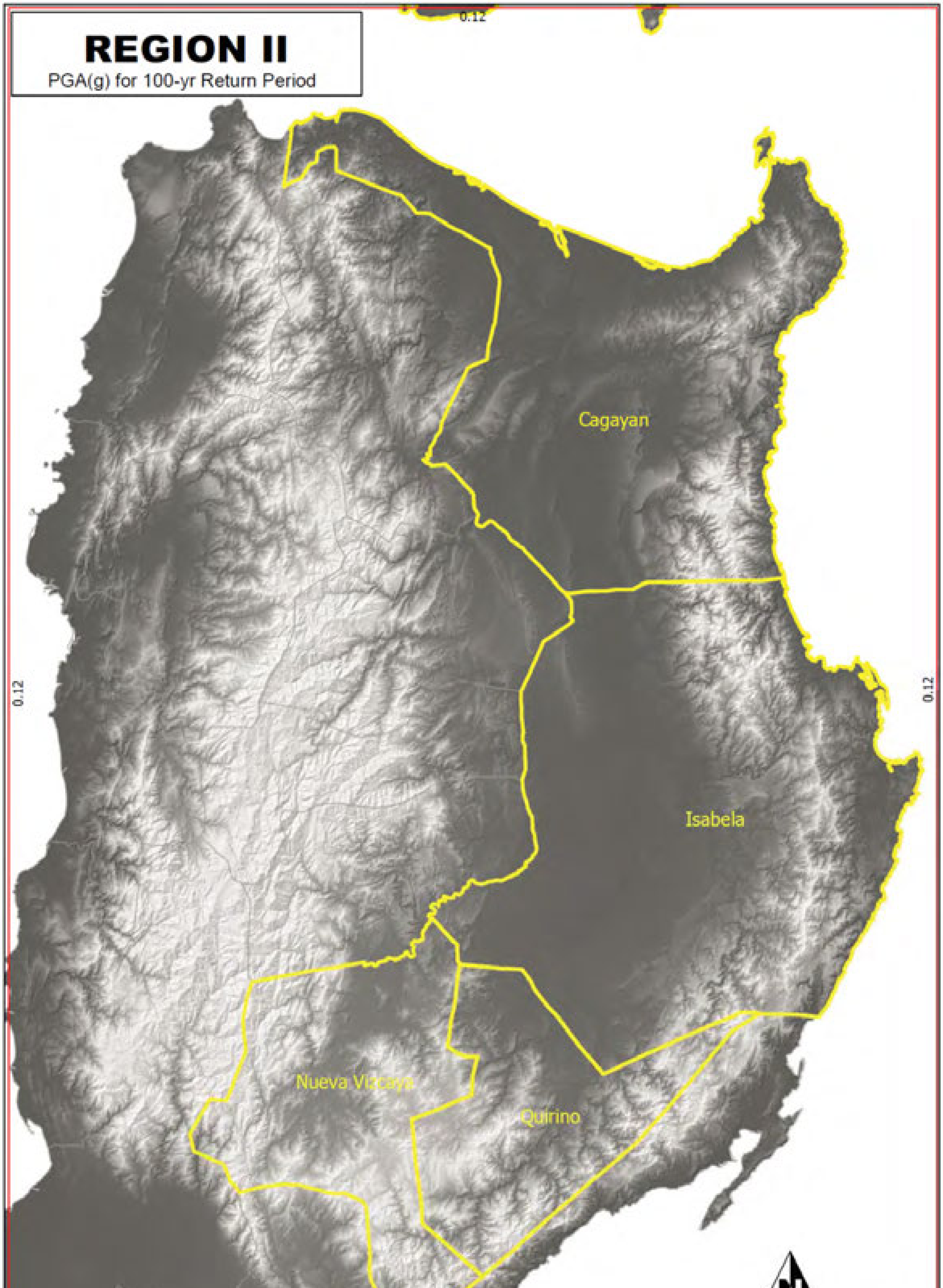


Figure 3A-3 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region II)

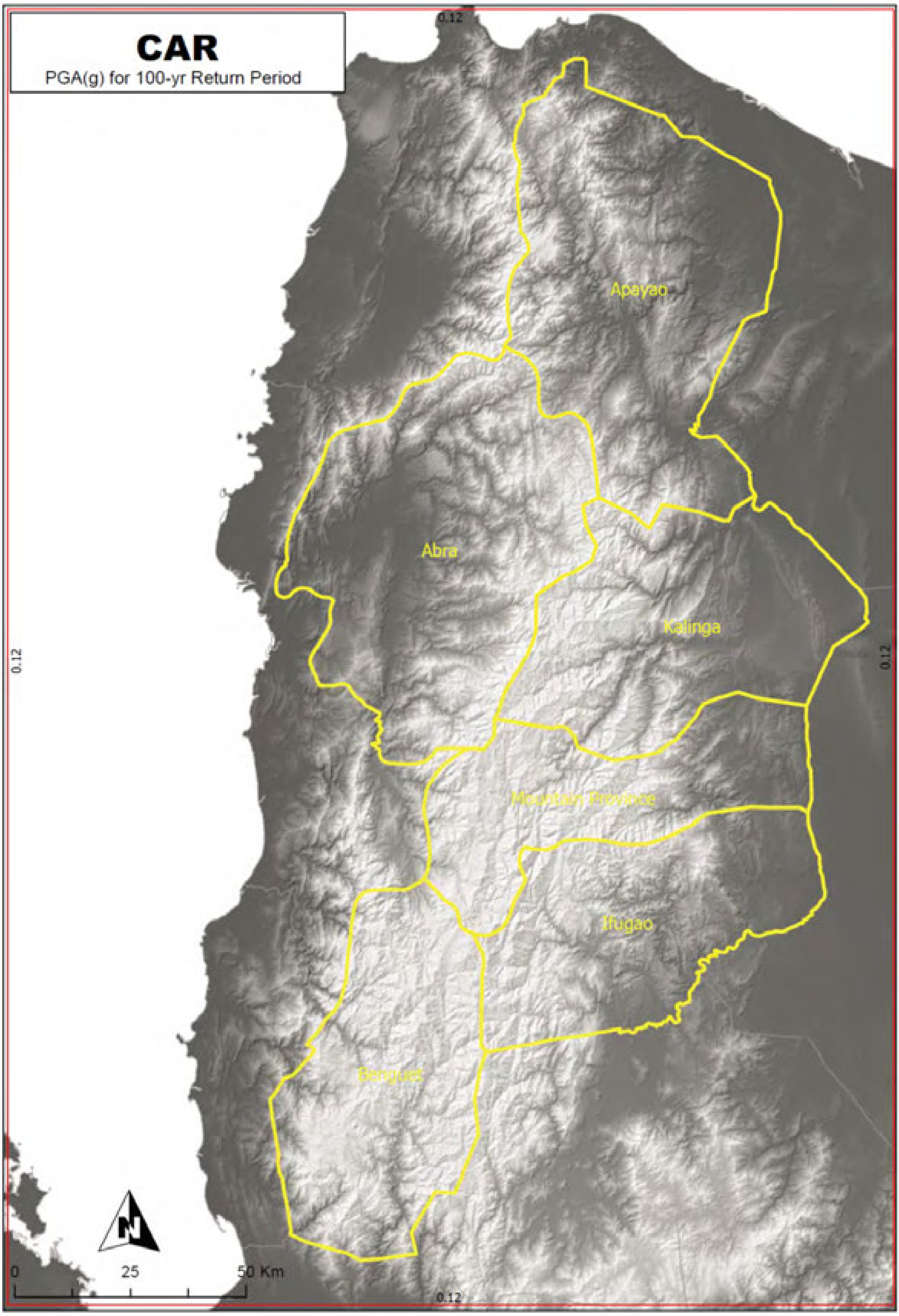


Figure 3A-4 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (CAR)

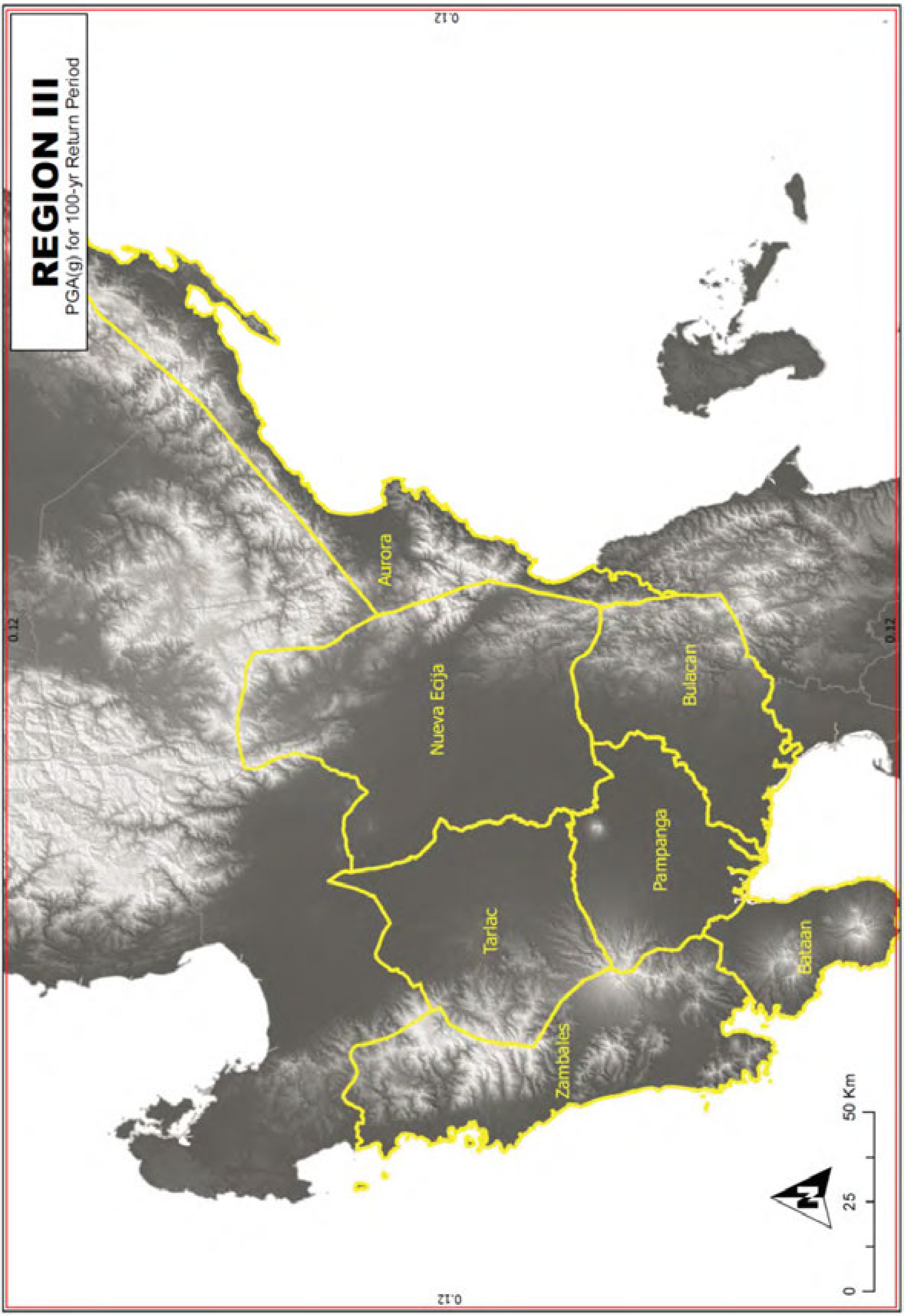


Figure 3A-5 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region III)

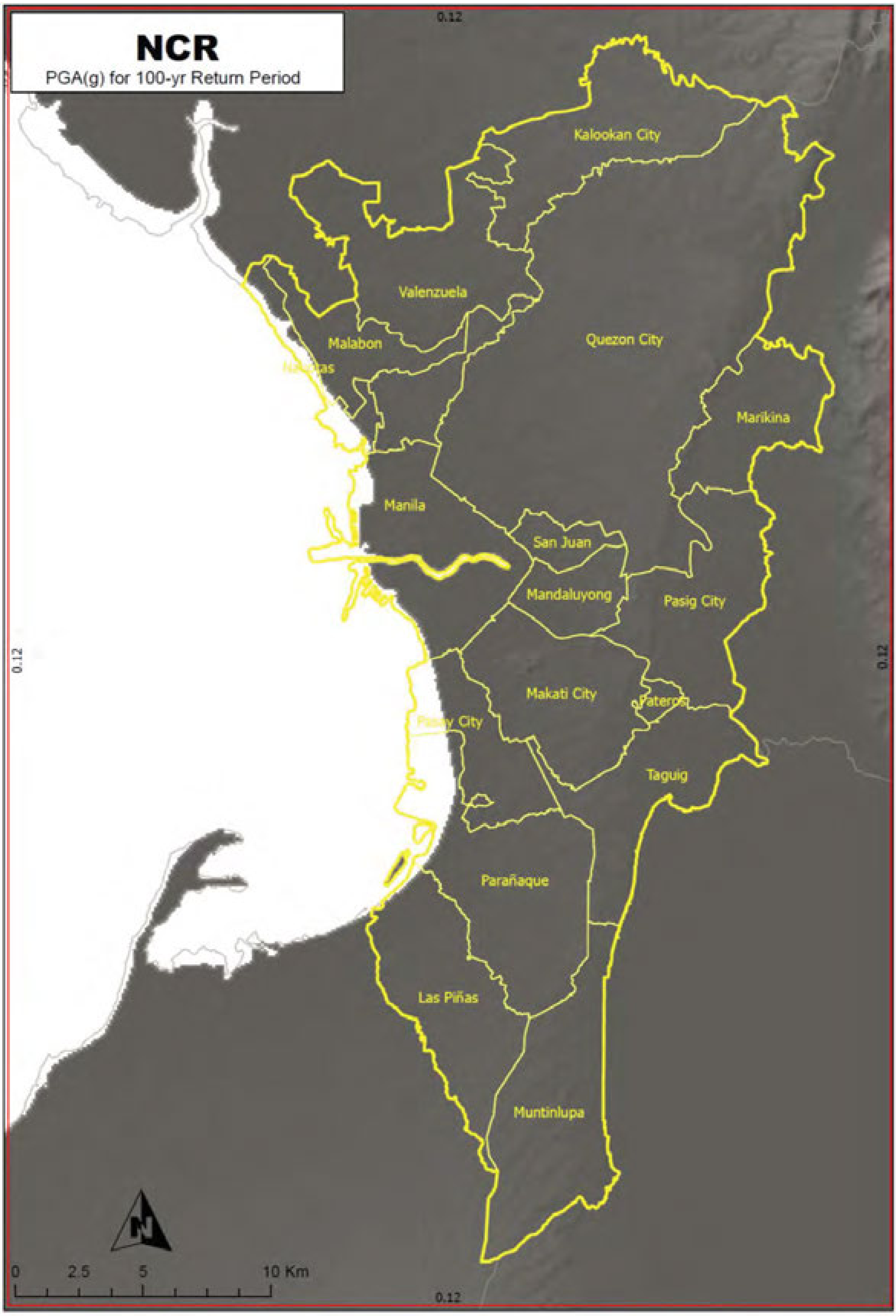


Figure 3A-6 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (NCR)

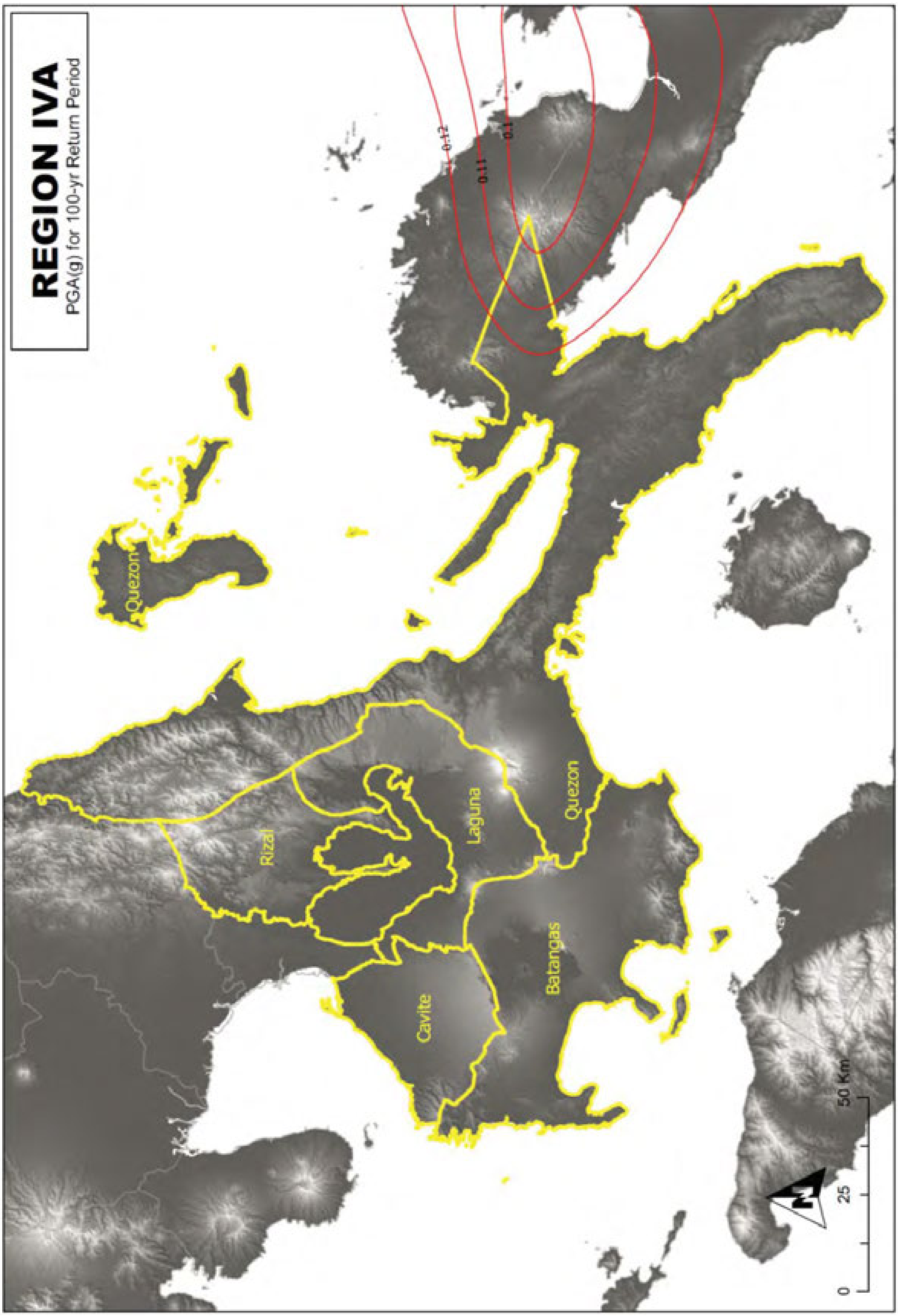


Figure 3A-7 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IV-A)

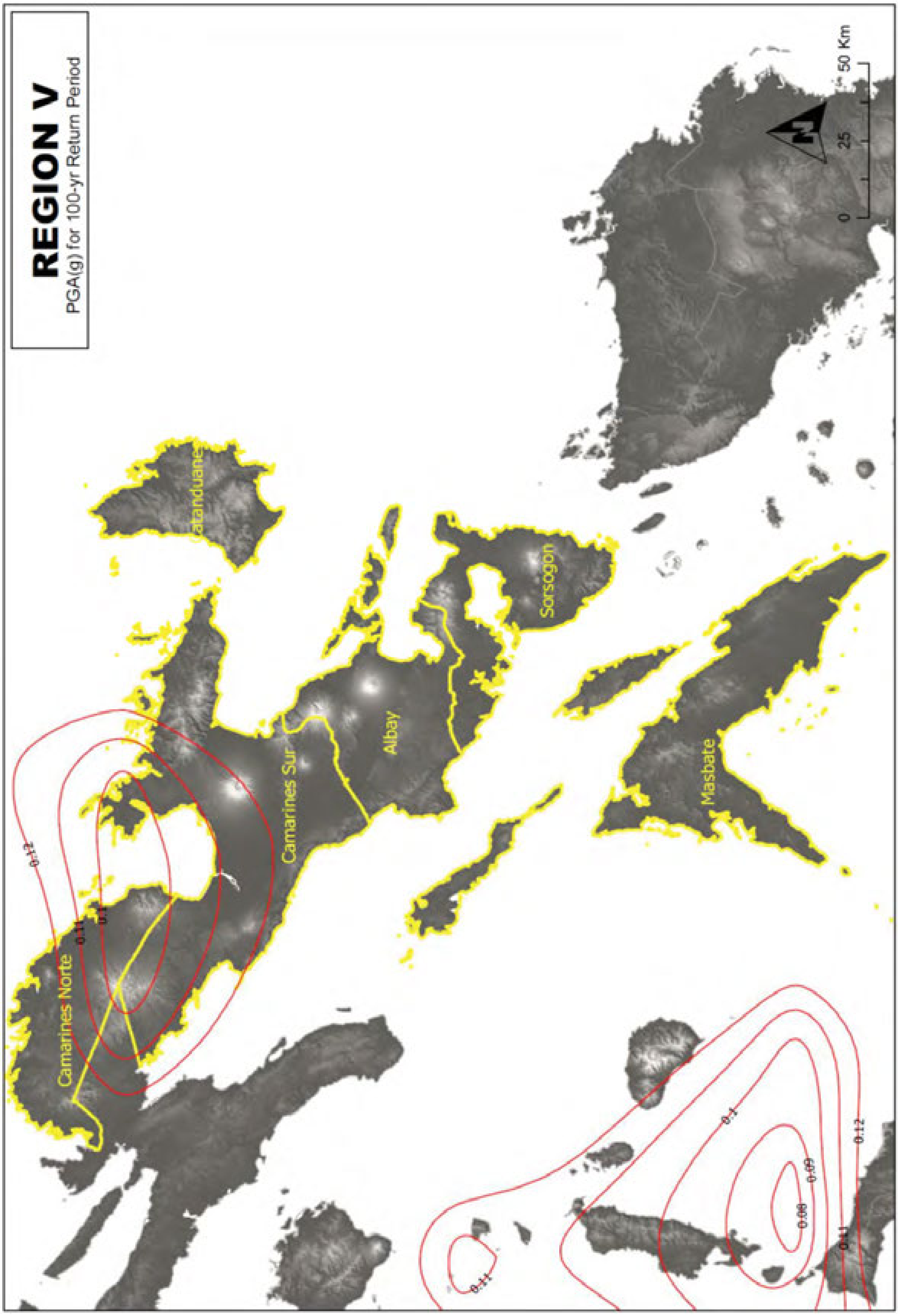


Figure 3A-8 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region V)

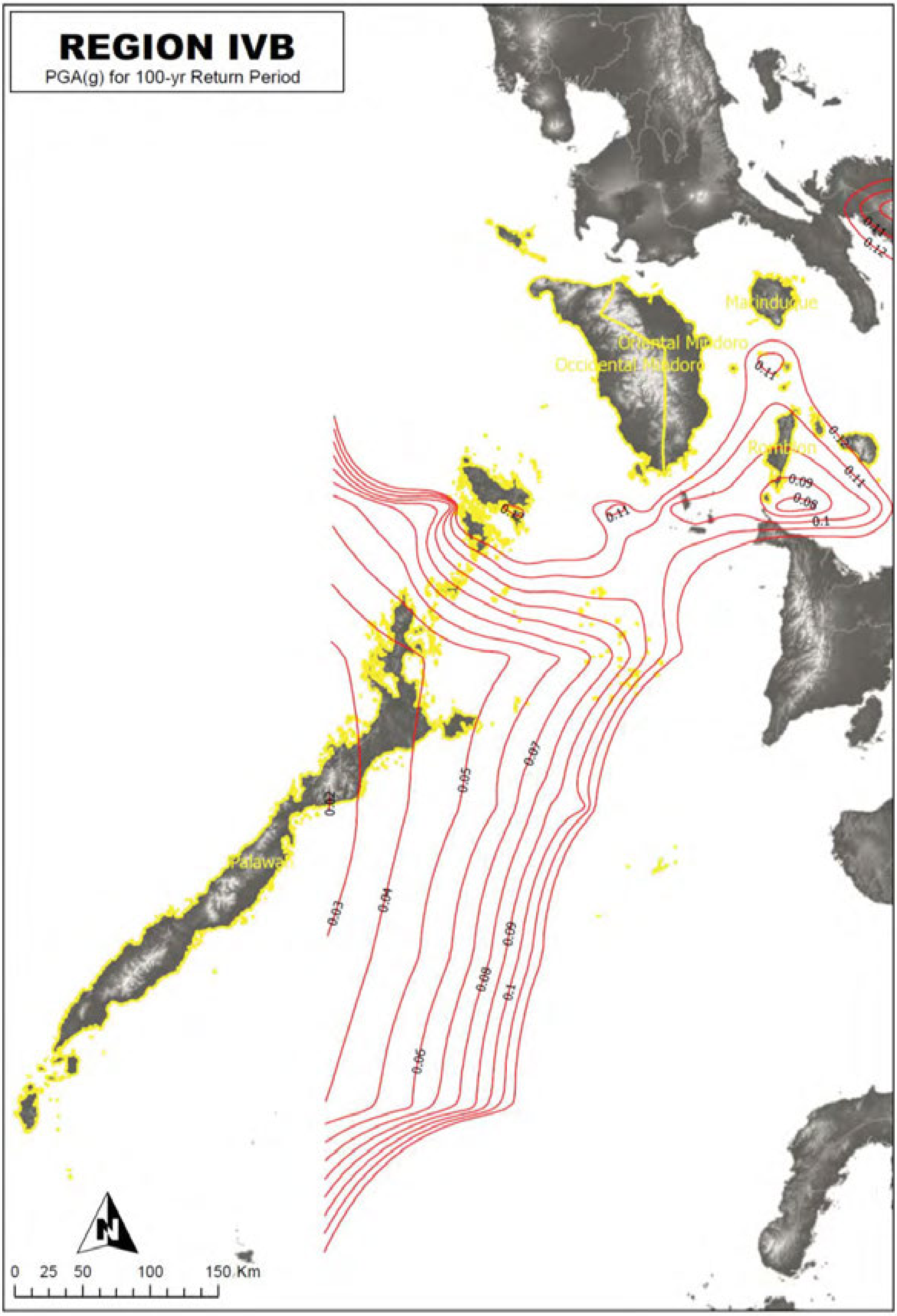


Figure 3A-9 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IV-B)

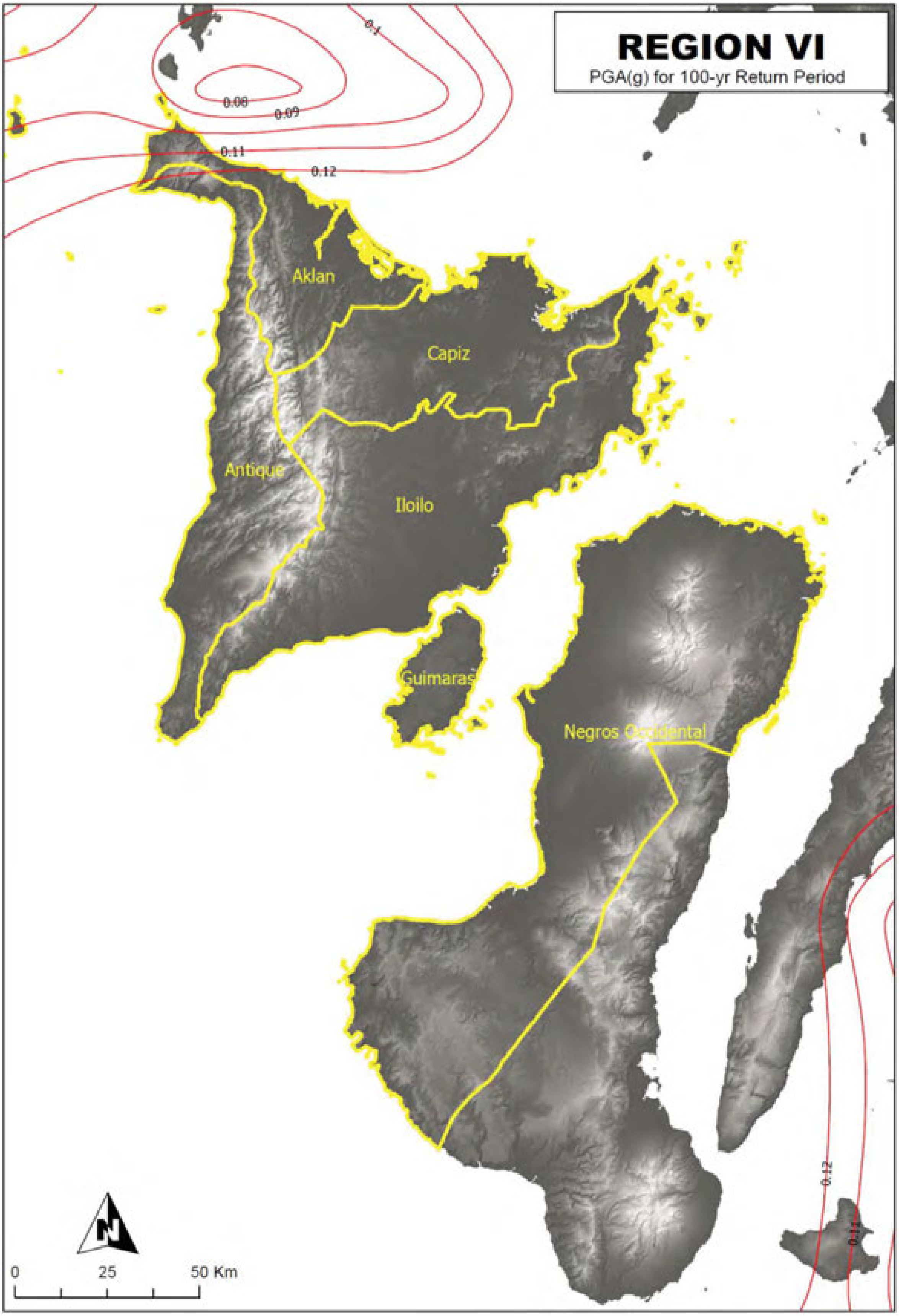


Figure 3A-10 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VI)

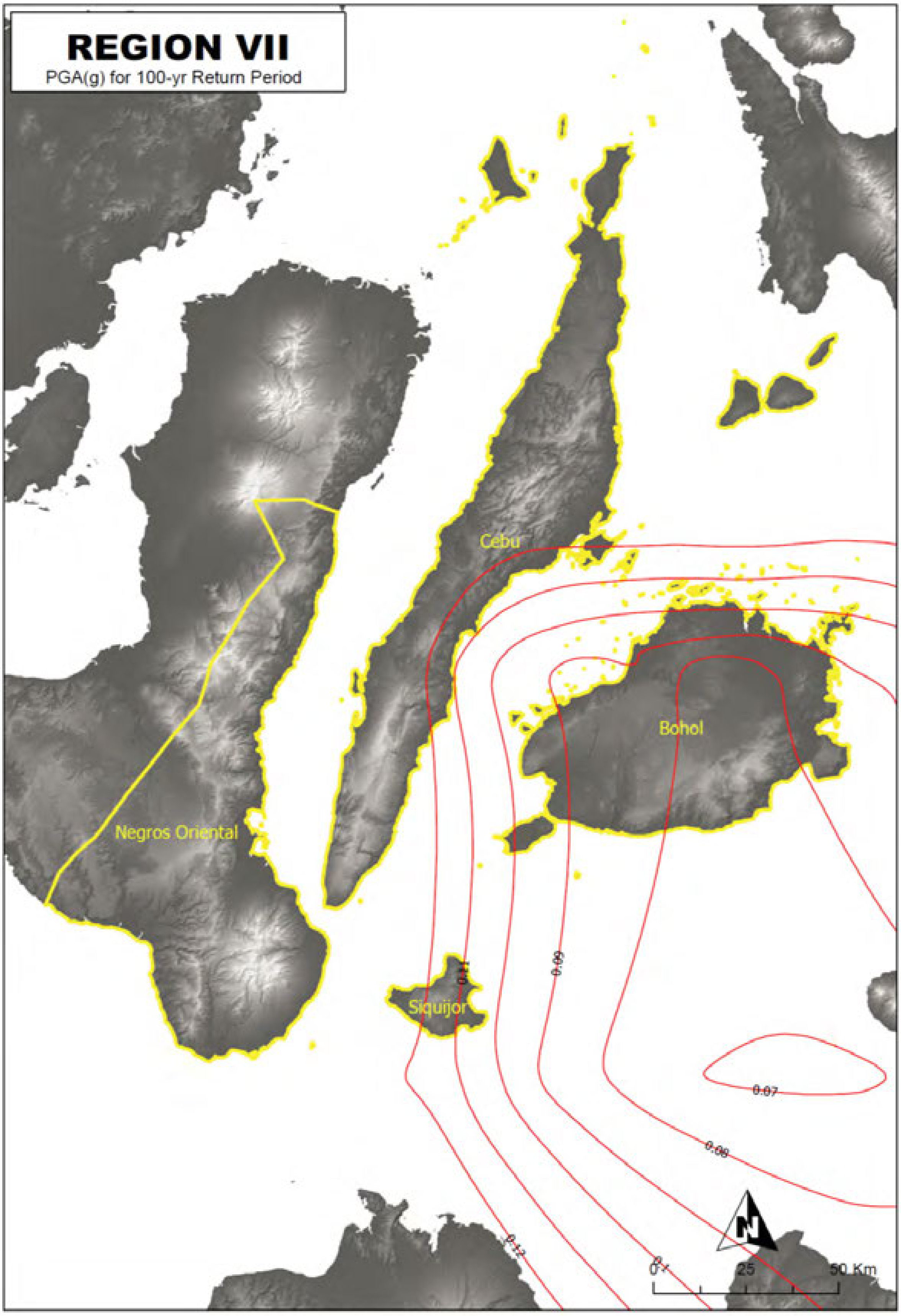


Figure 3A-11 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VII)

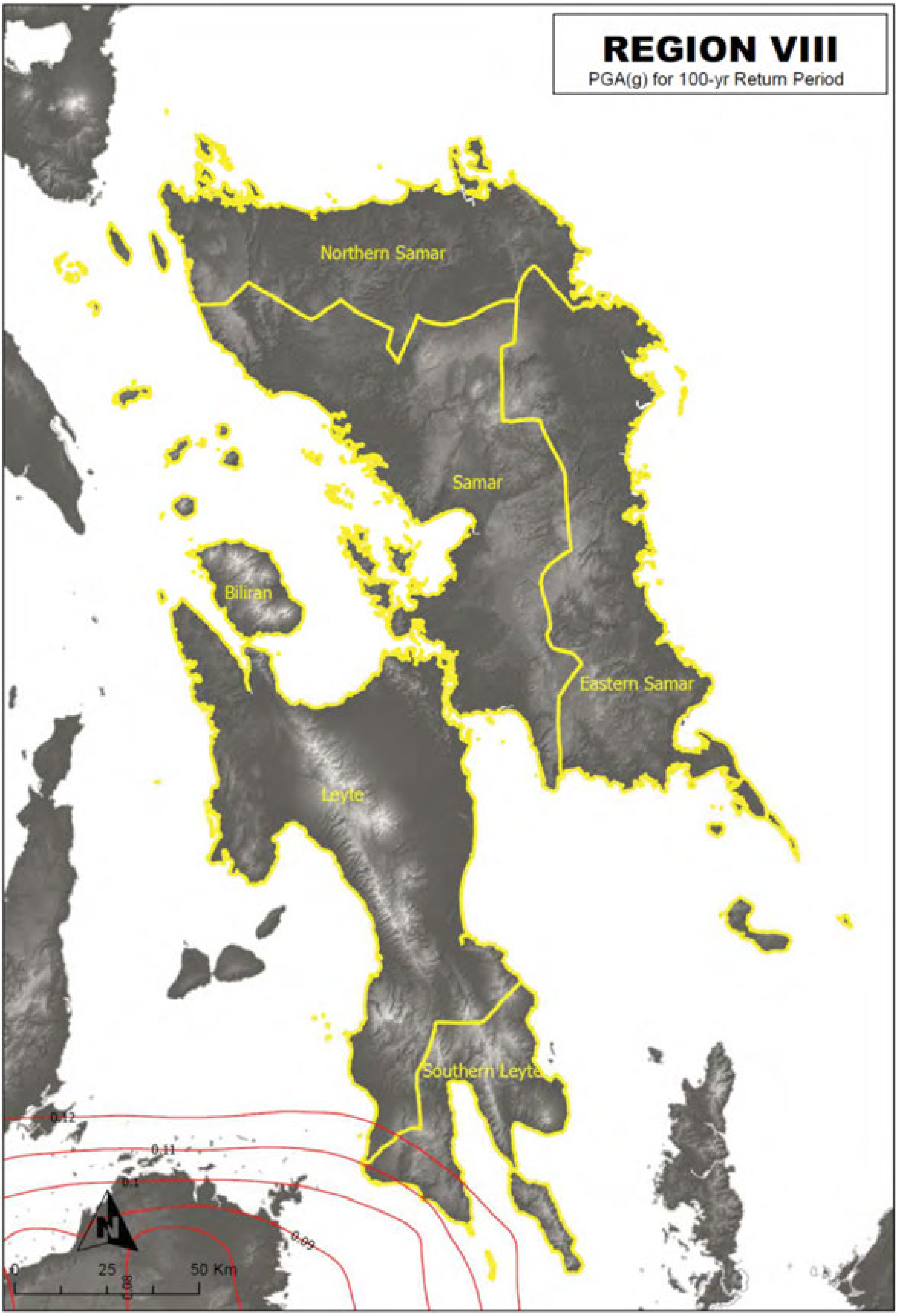


Figure 3A-12 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region VIII)

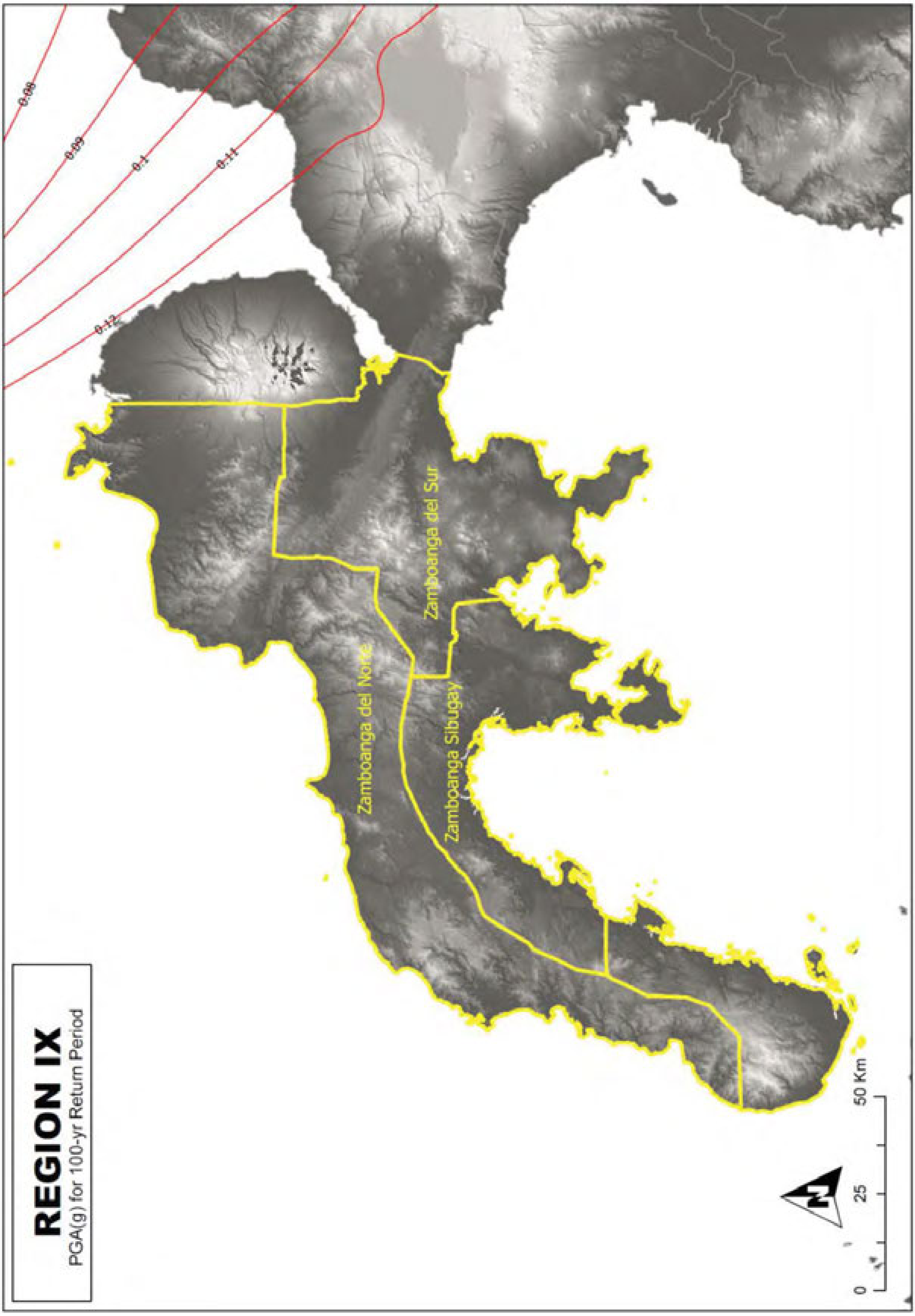


Figure 3A-13 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region IX)

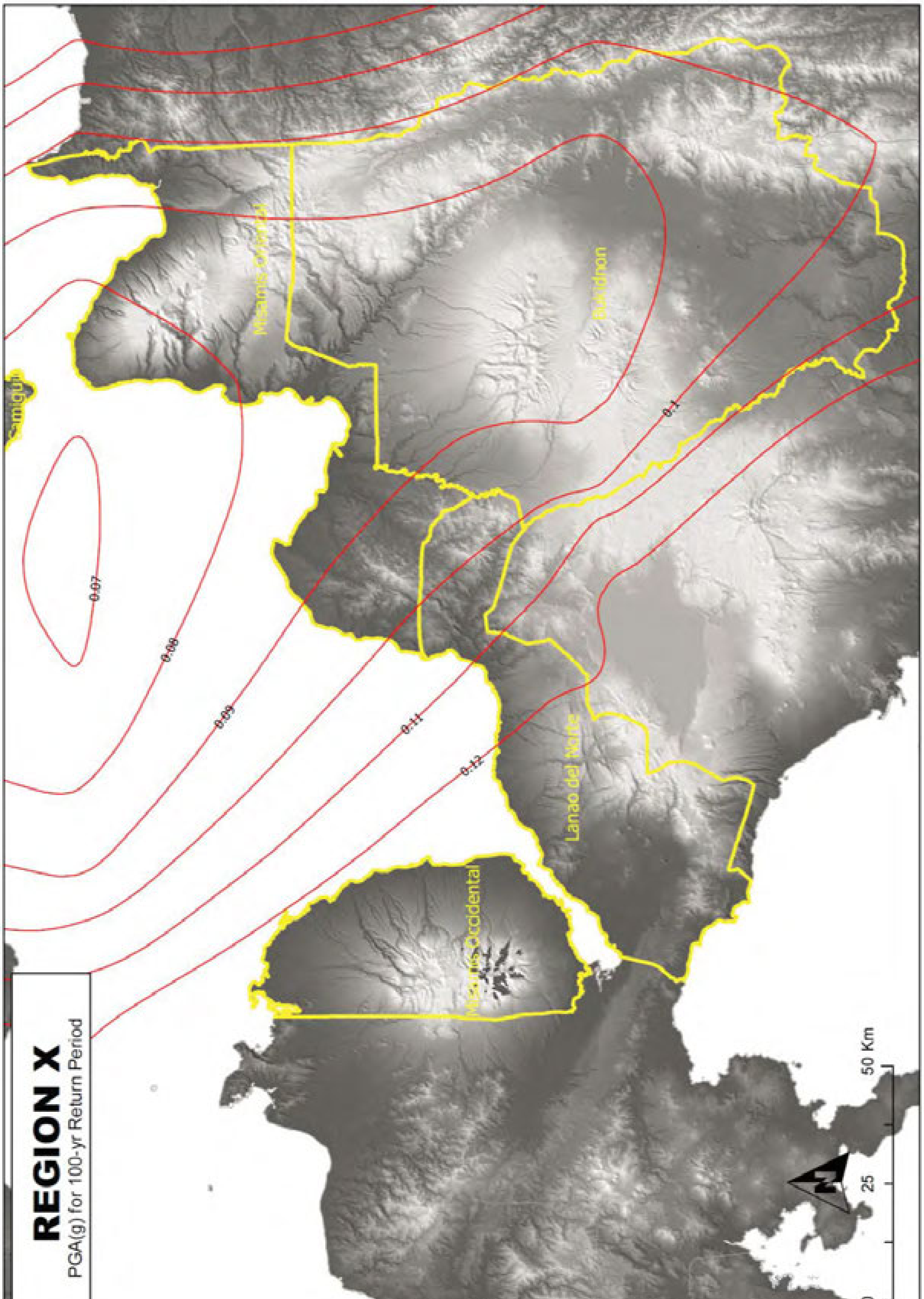


Figure 3A-14 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region X)

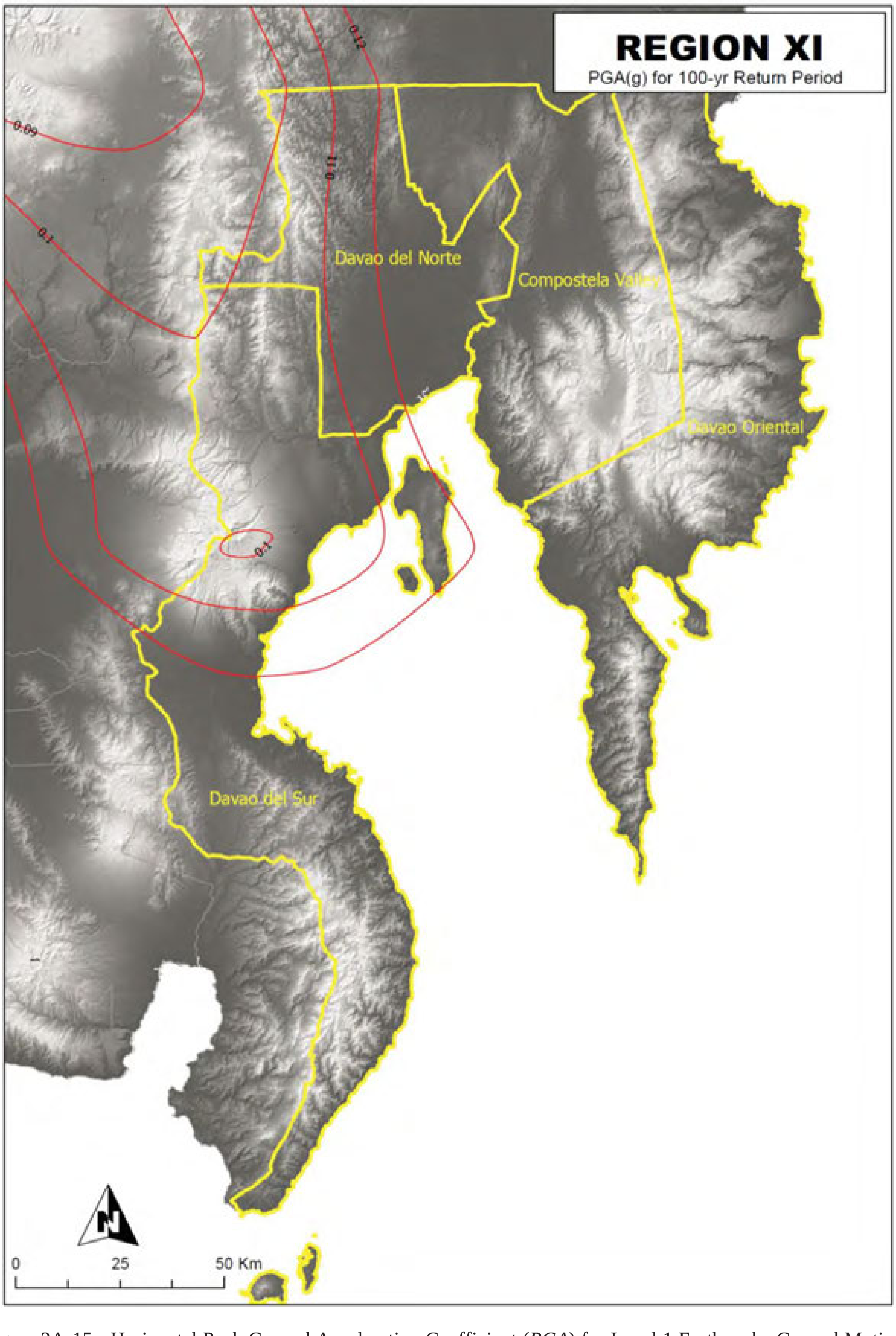


Figure 3A-15 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XI)

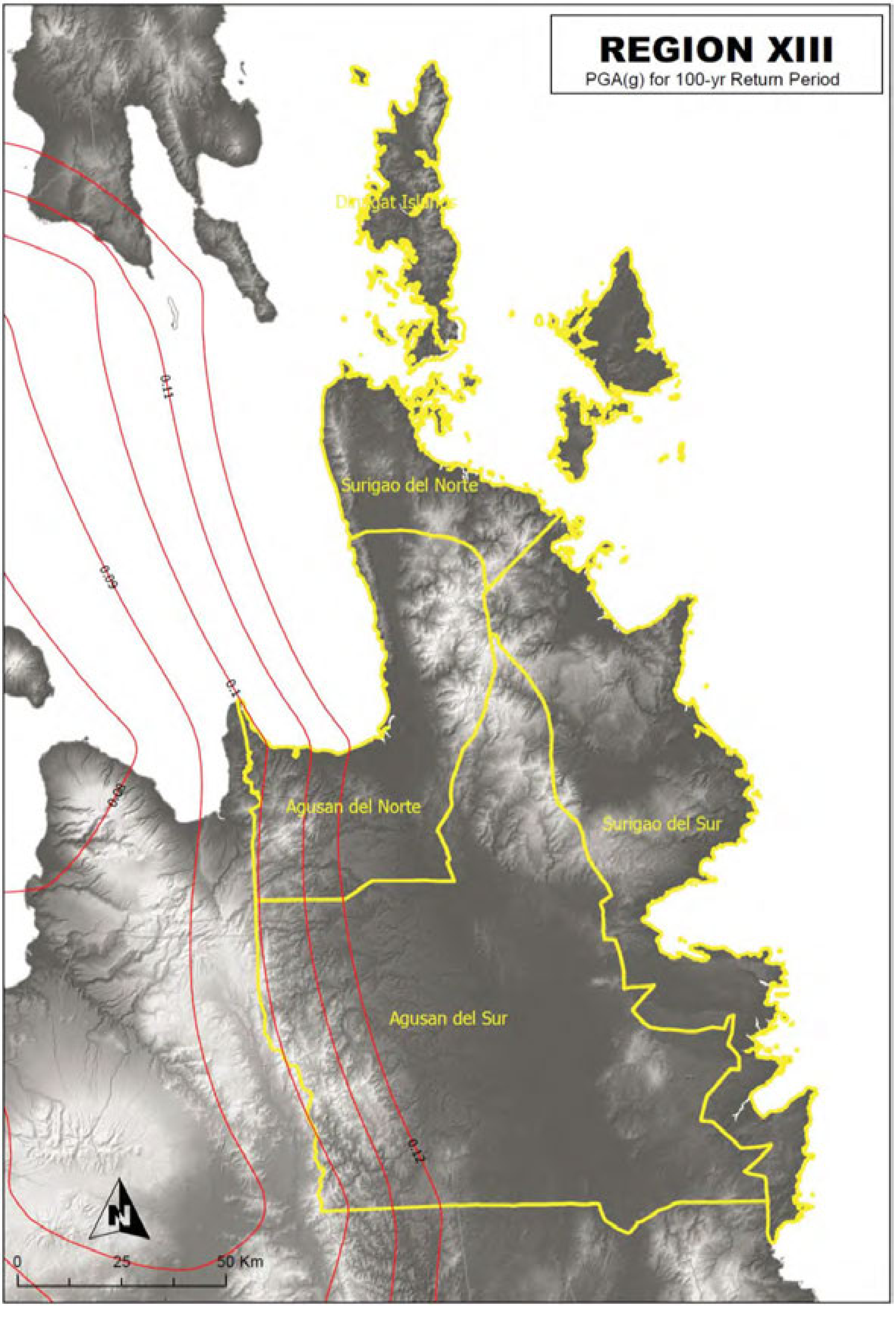


Figure 3A-16 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XIII)

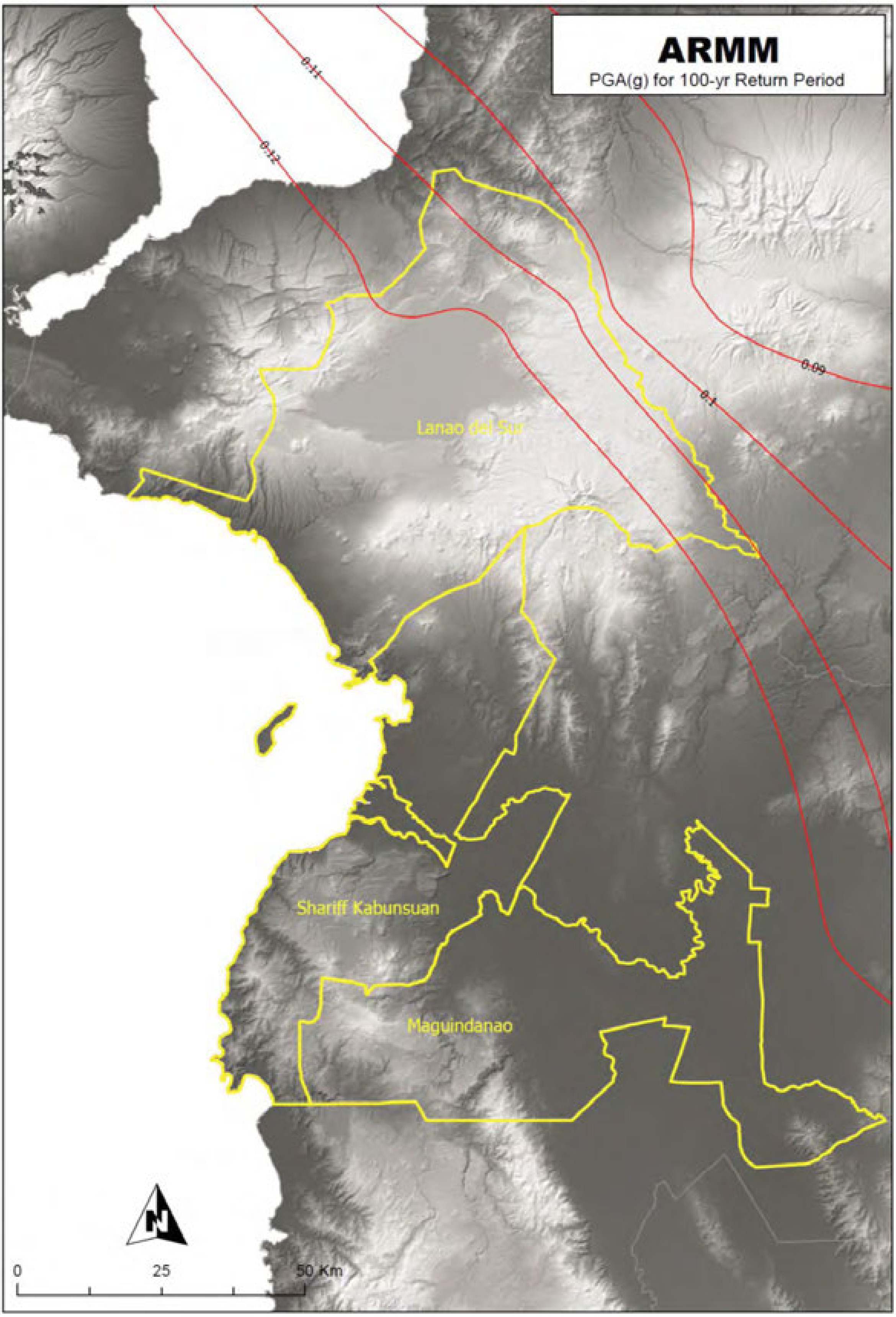


Figure 3A-17 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region ARMM)

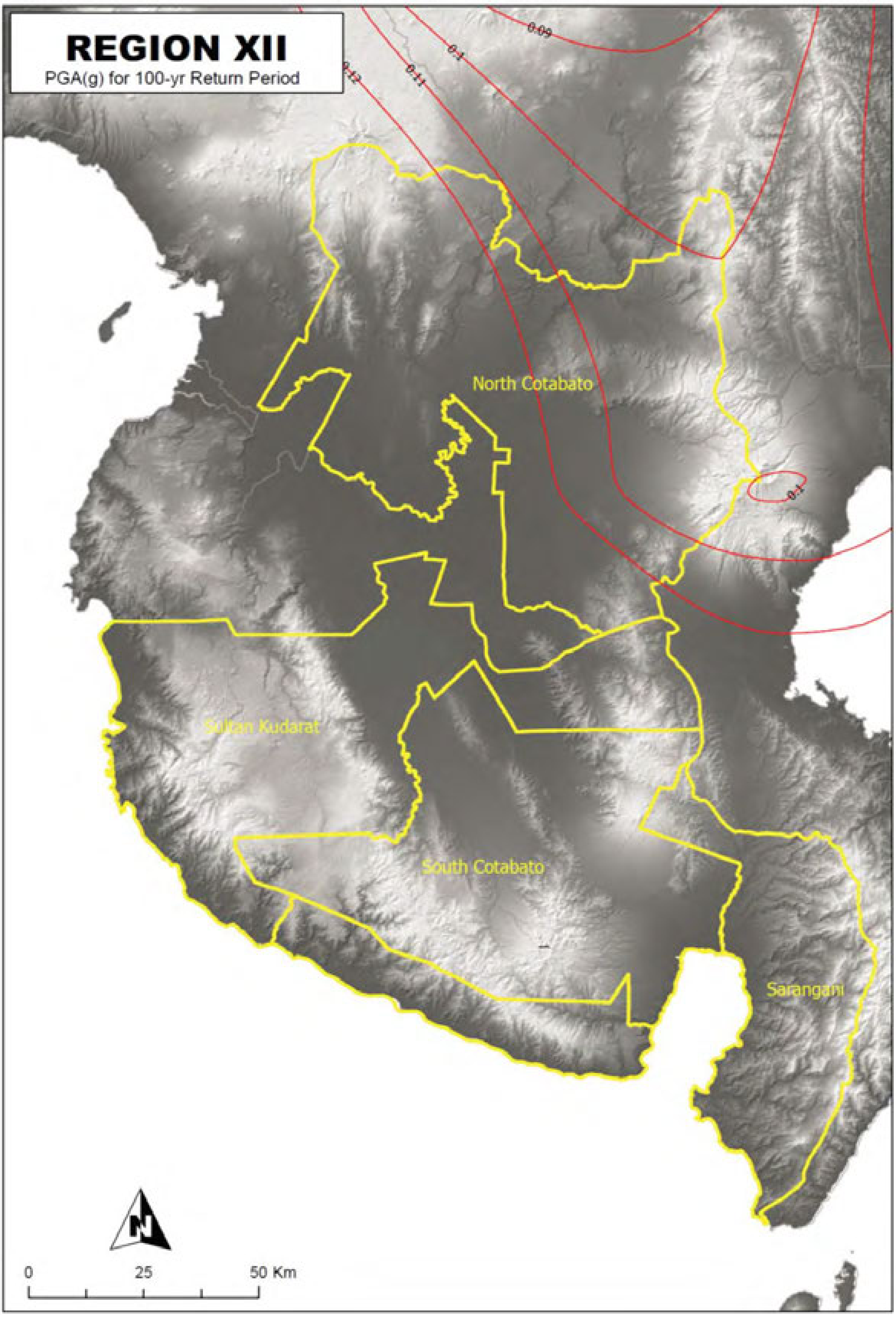


Figure 3A-18 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level 1 Earthquake Ground Motion (Region XII)

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT 0.20-SEC PERIOD

(100-YEAR RETURN PERIOD)

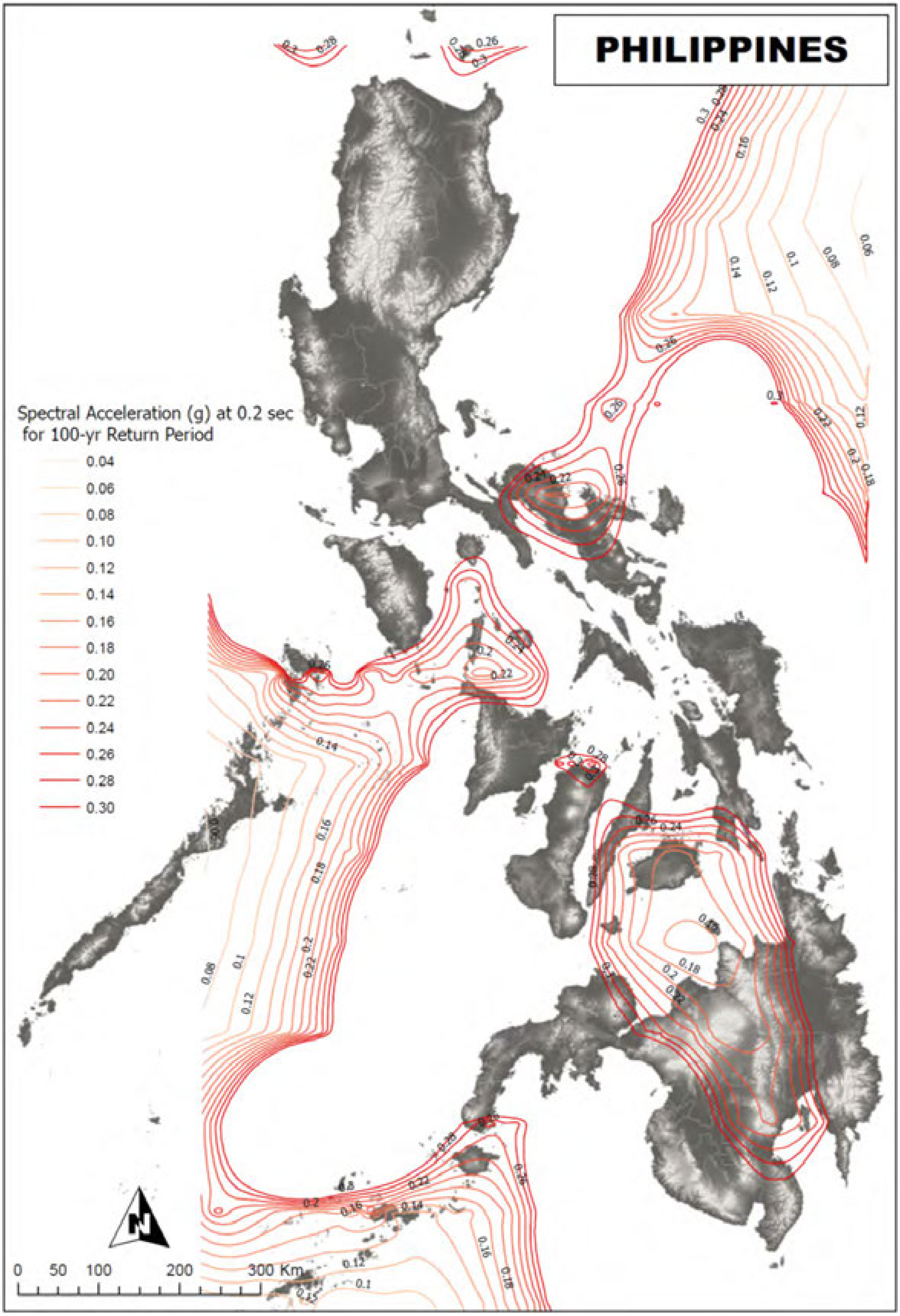


Figure 3A-19 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Entire Country)

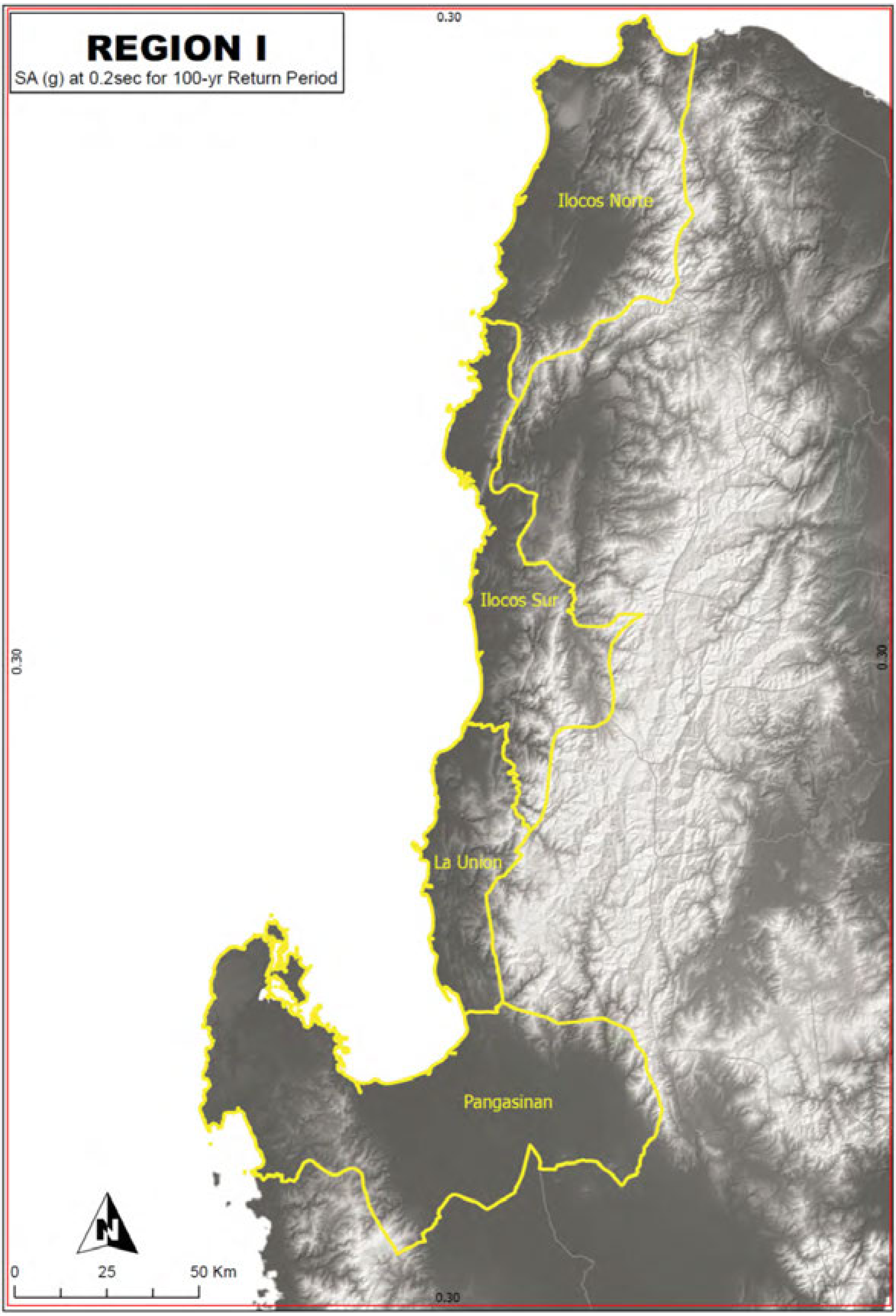


Figure 3A-20 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) Level 1 Earthquake Ground Motion (Region I)

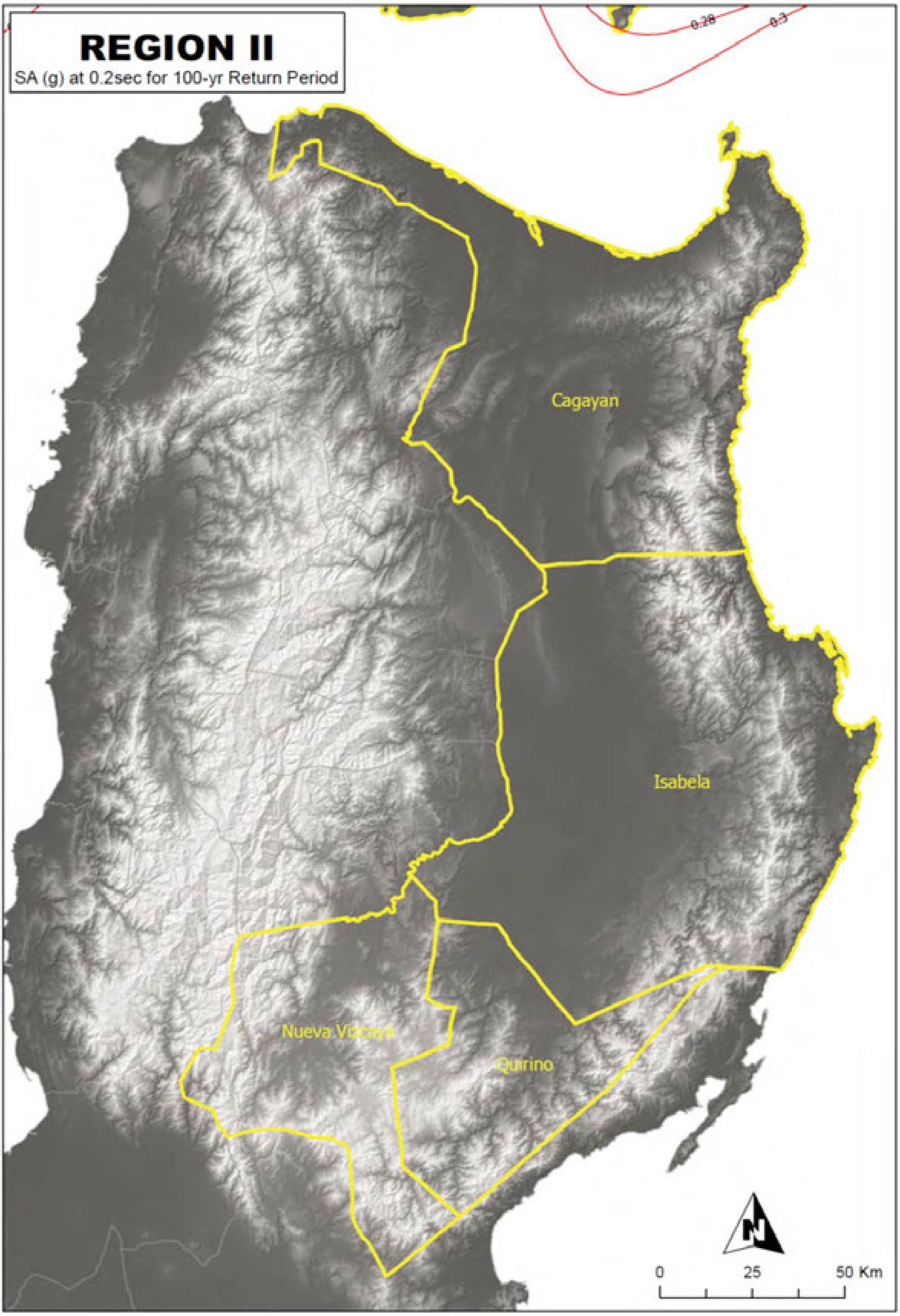


Figure 3A-21 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region II)

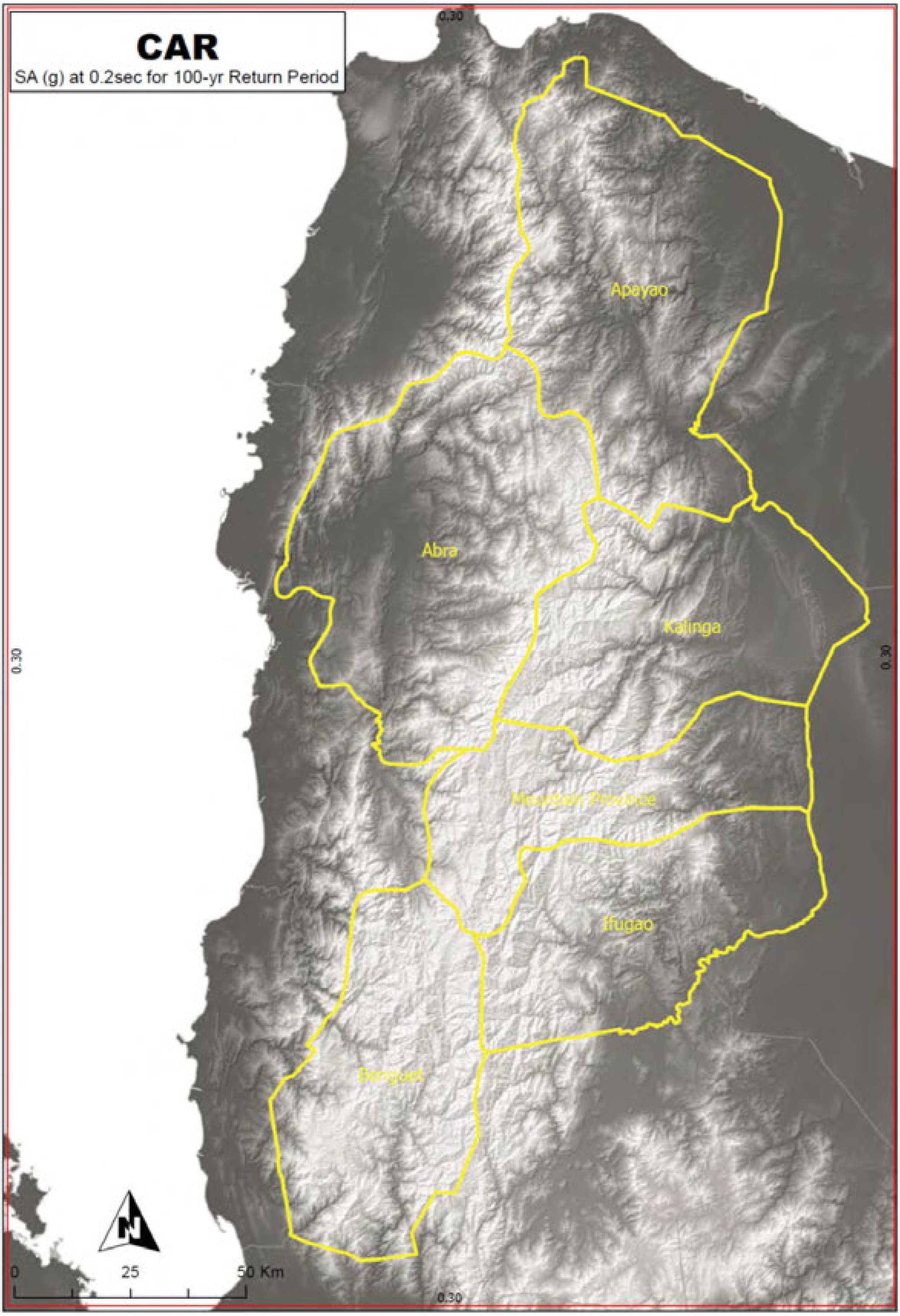


Figure 3A-22 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (CAR)

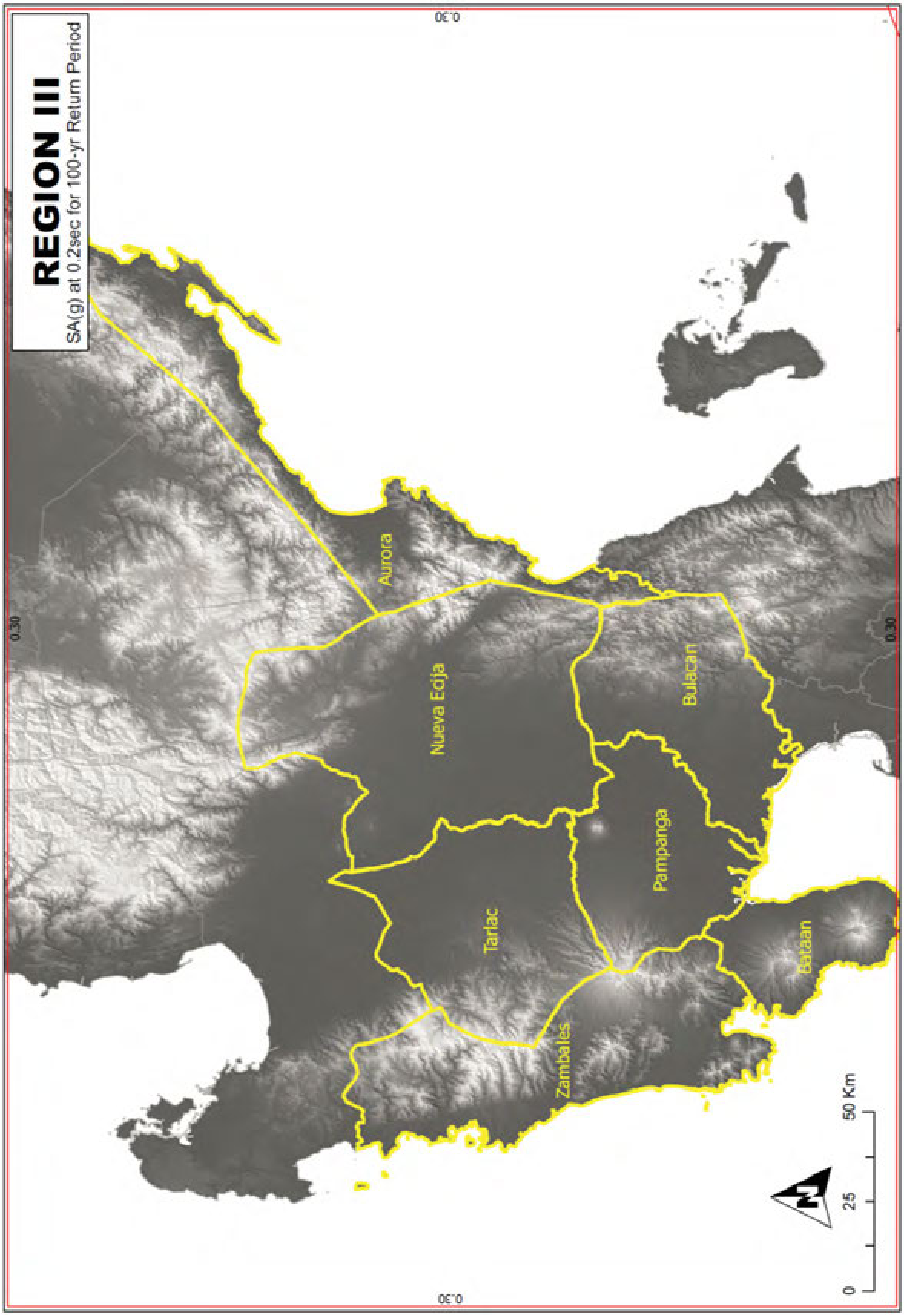


Figure 3A-23 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region III)

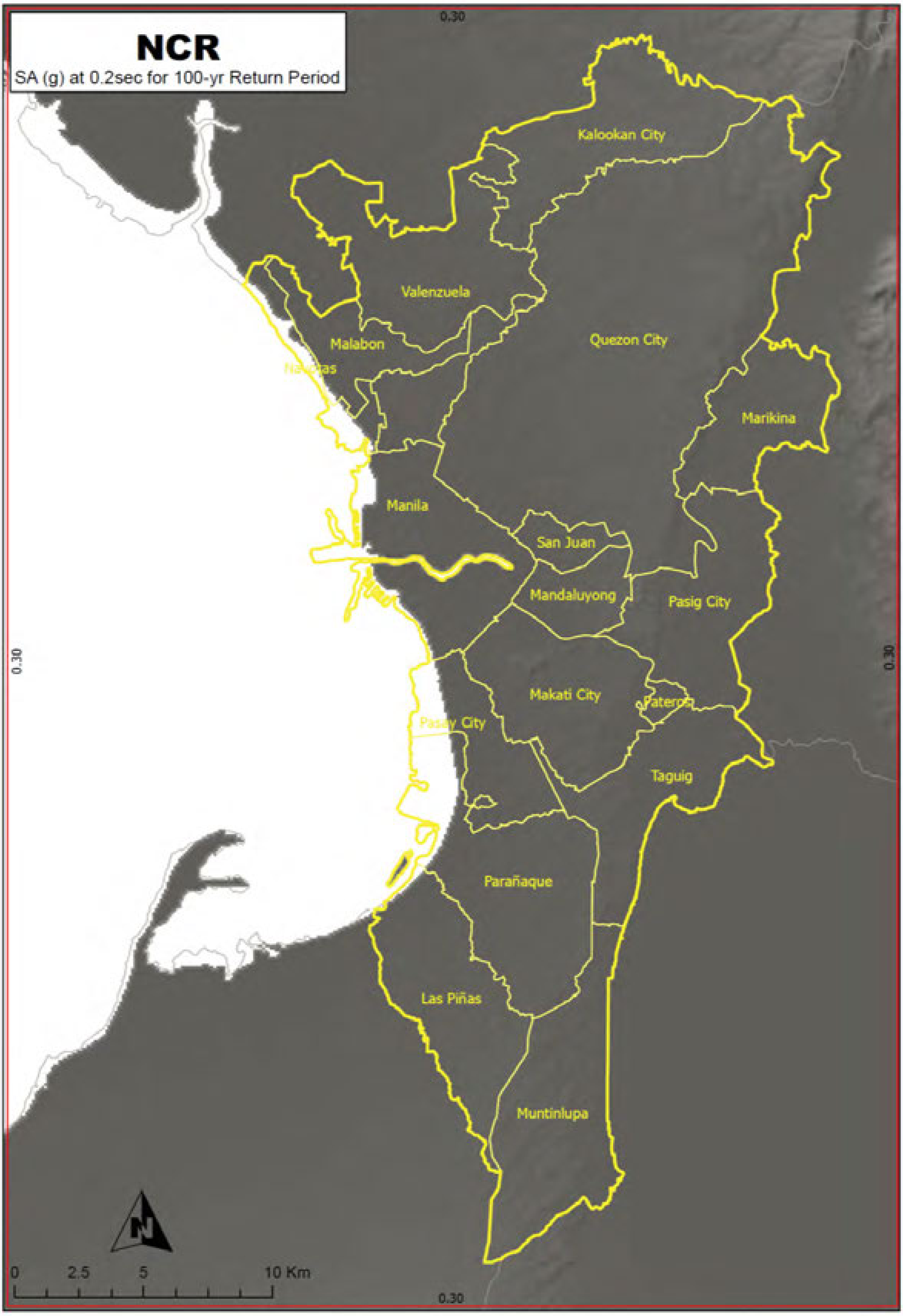


Figure 3A-24 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (NCR)

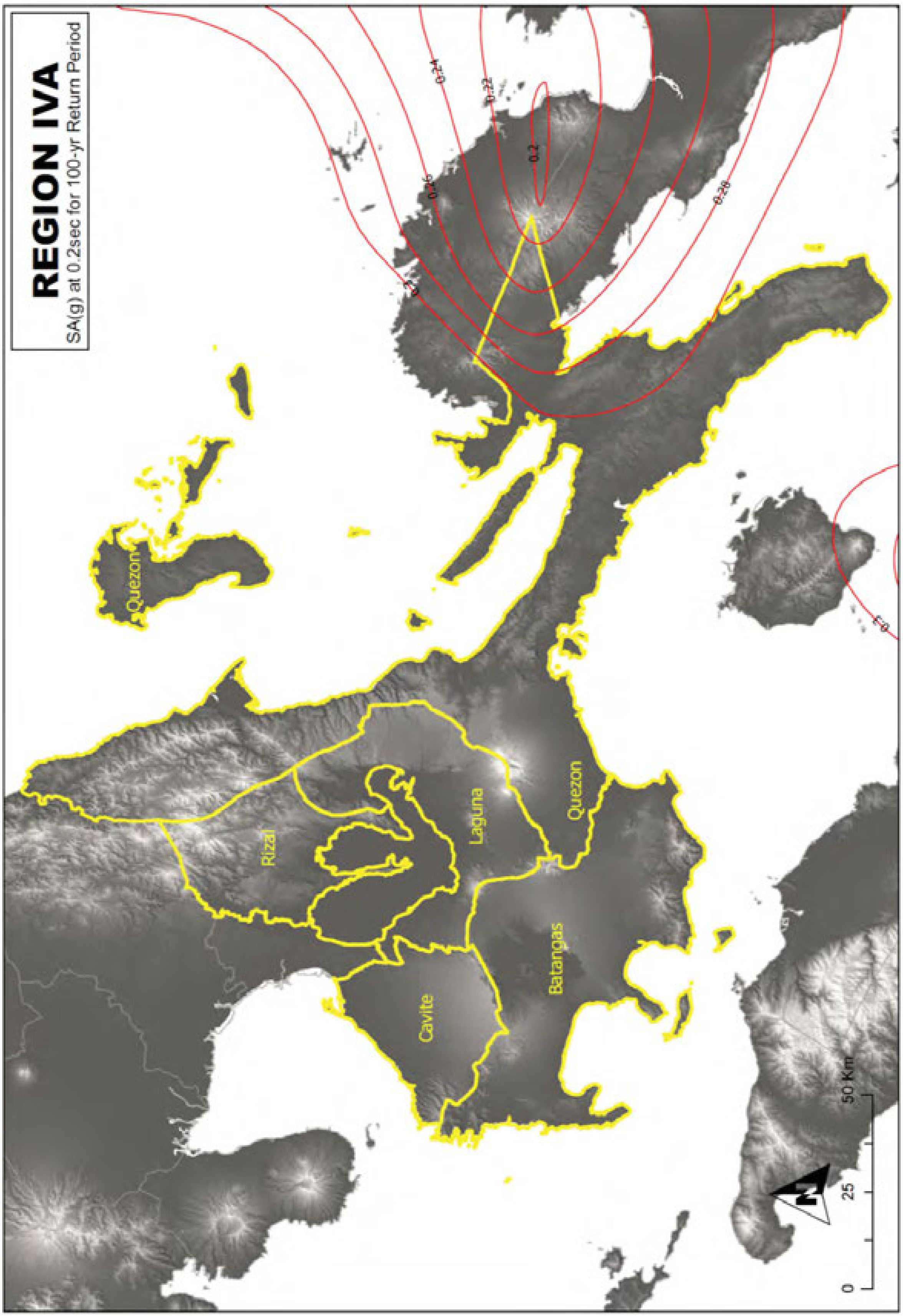


Figure 3A-25 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IV-A)

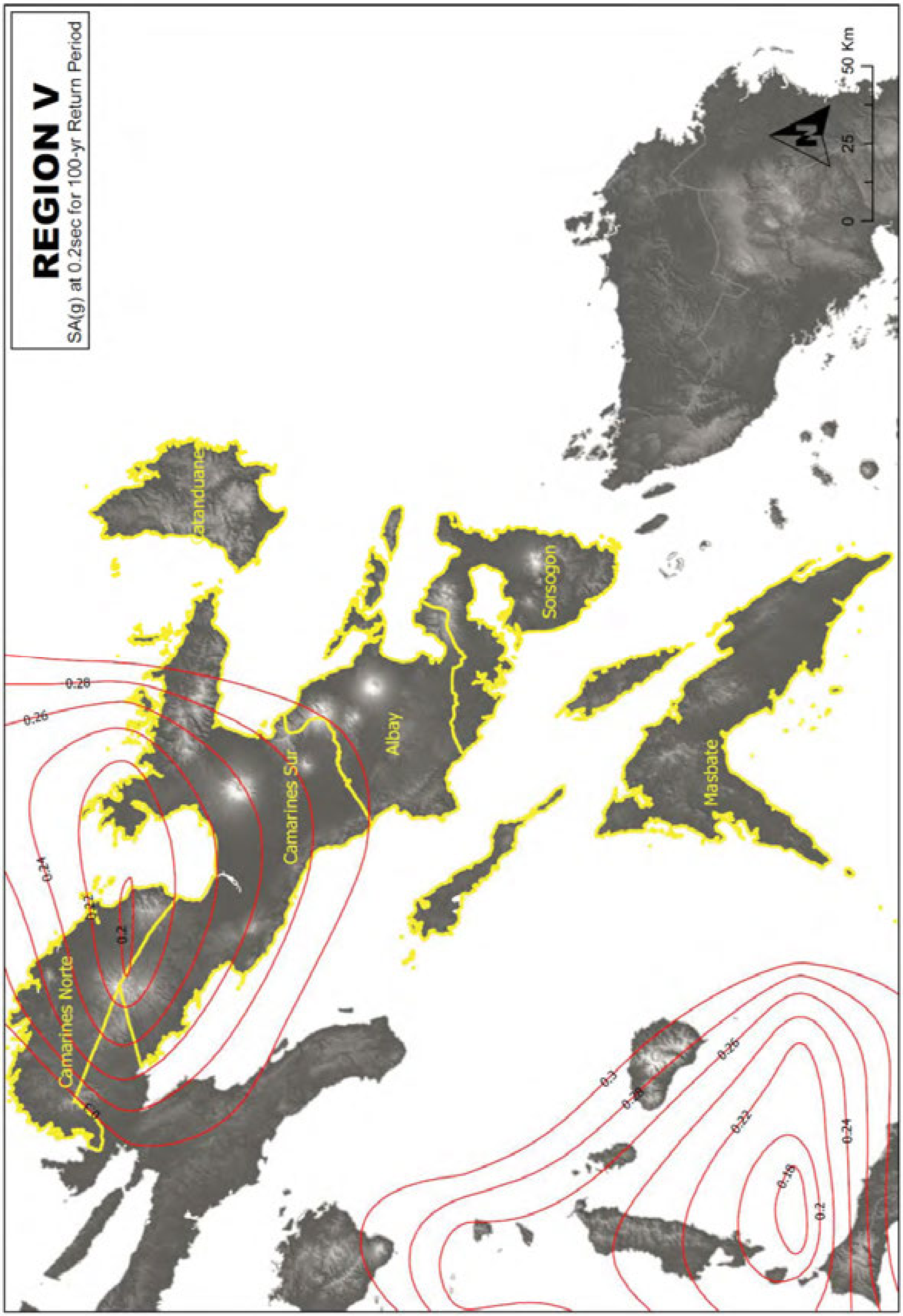


Figure 3A-26 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region V)

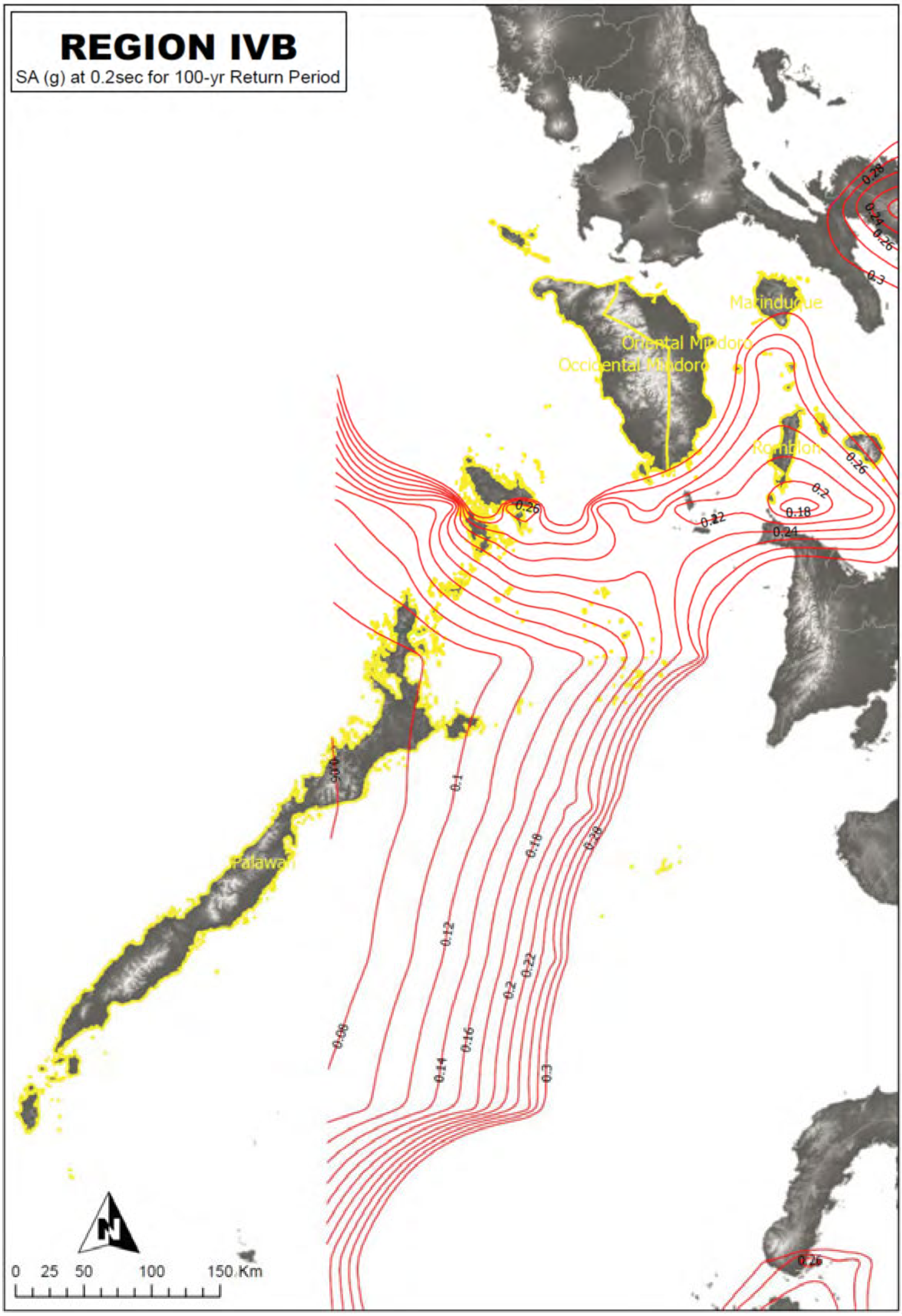


Figure 3A-27 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IV-B)

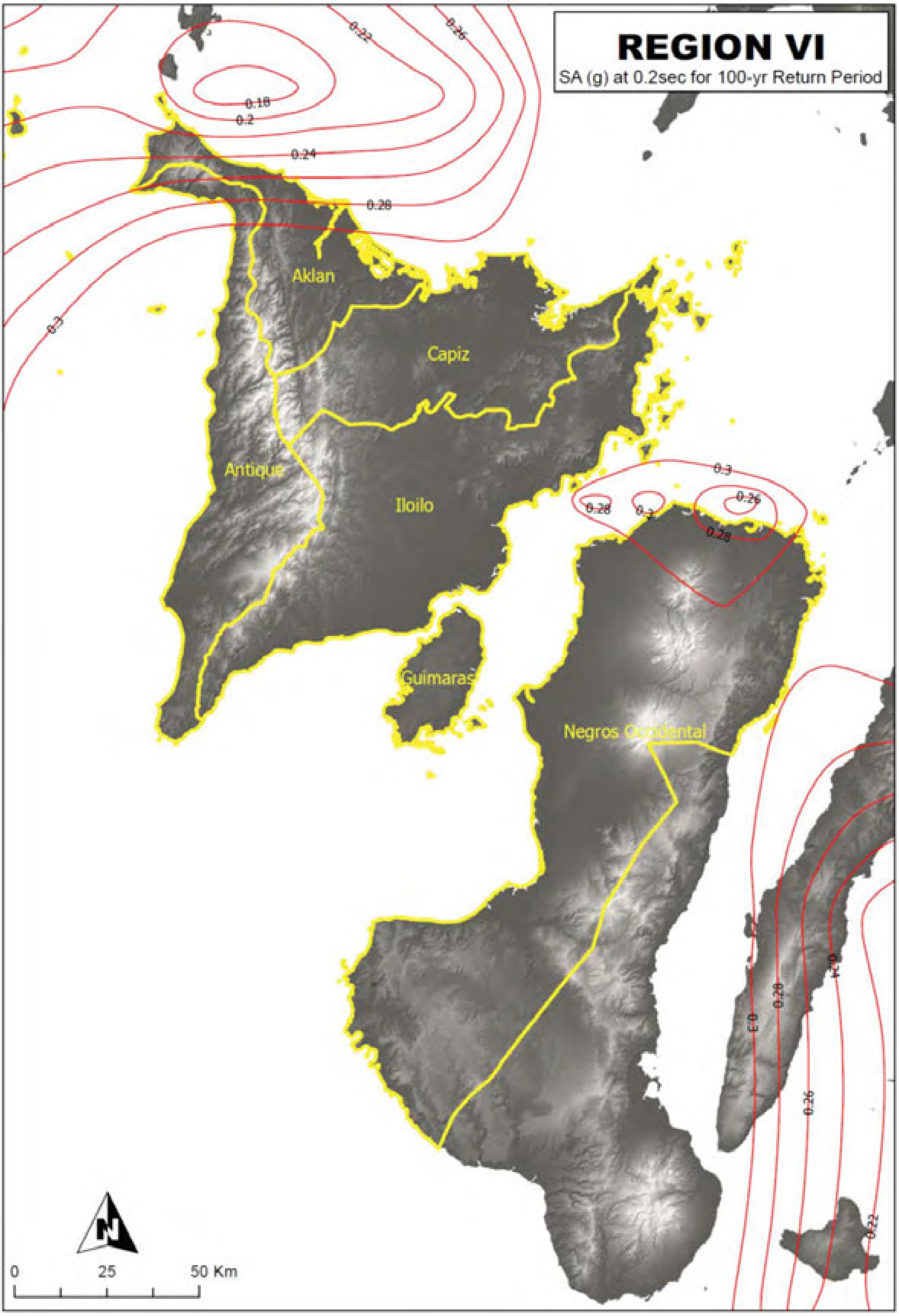


Figure 3A-28 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VI)

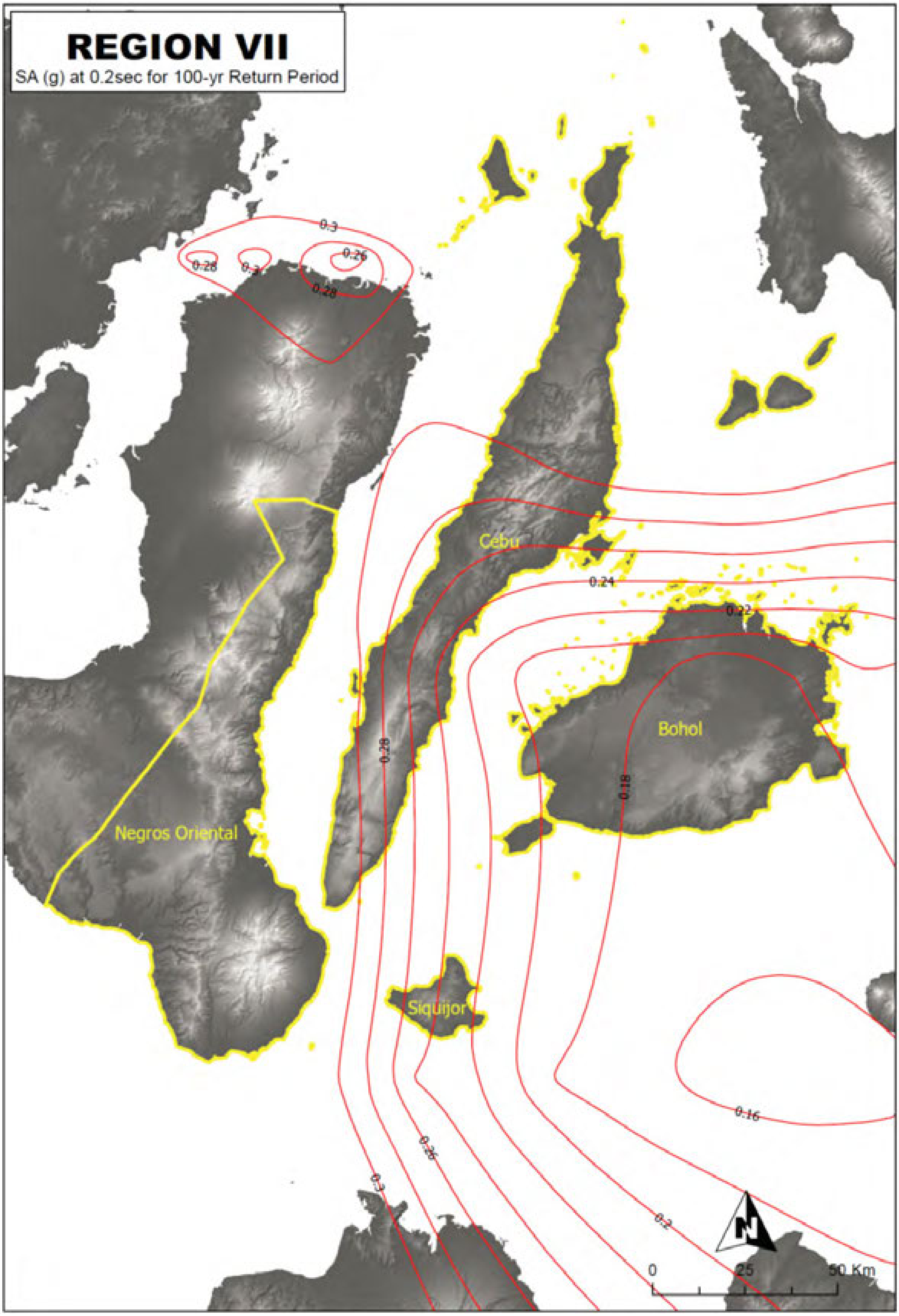


Figure 3A-29 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VII)

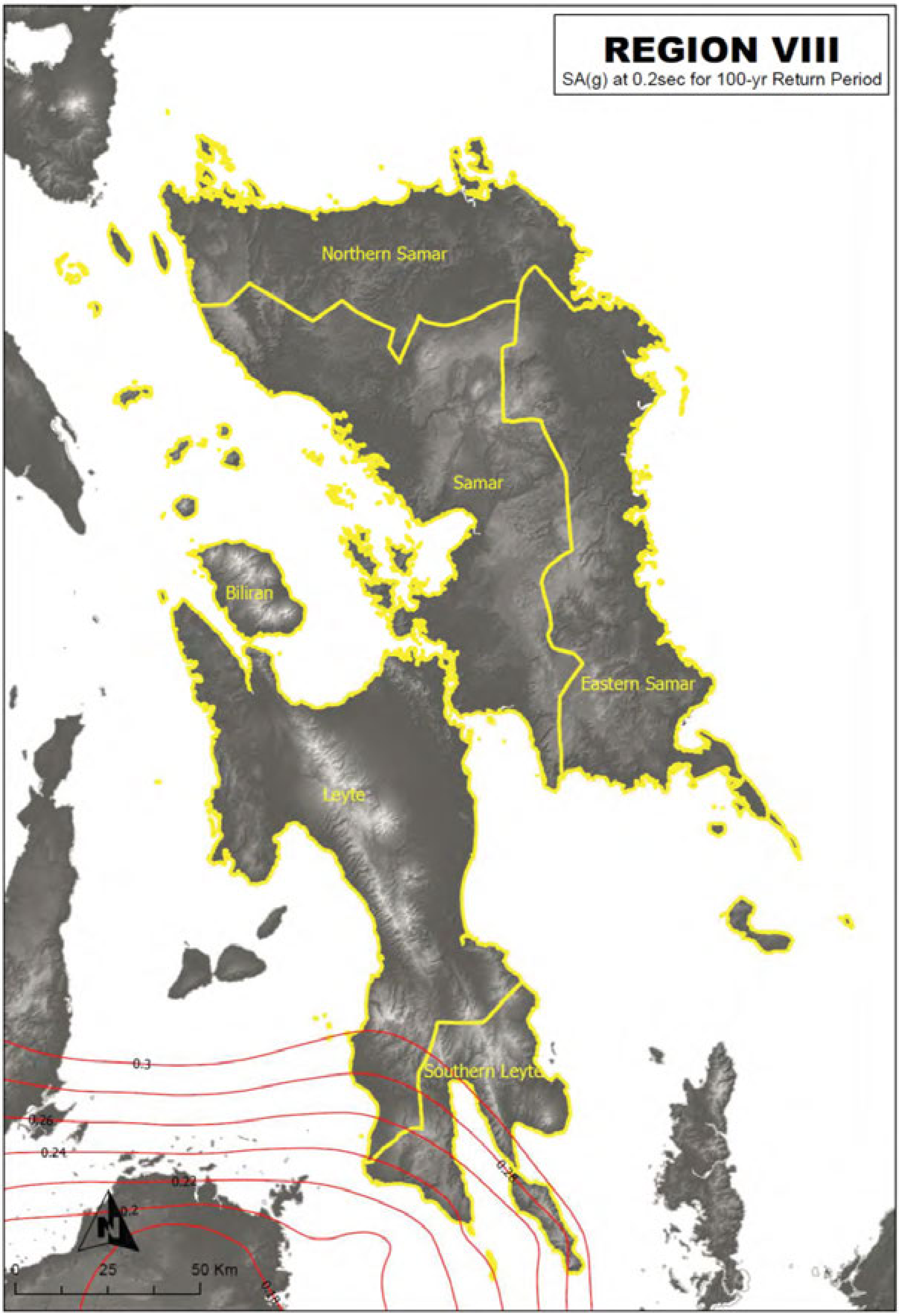


Figure 3A-30 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region VIII)

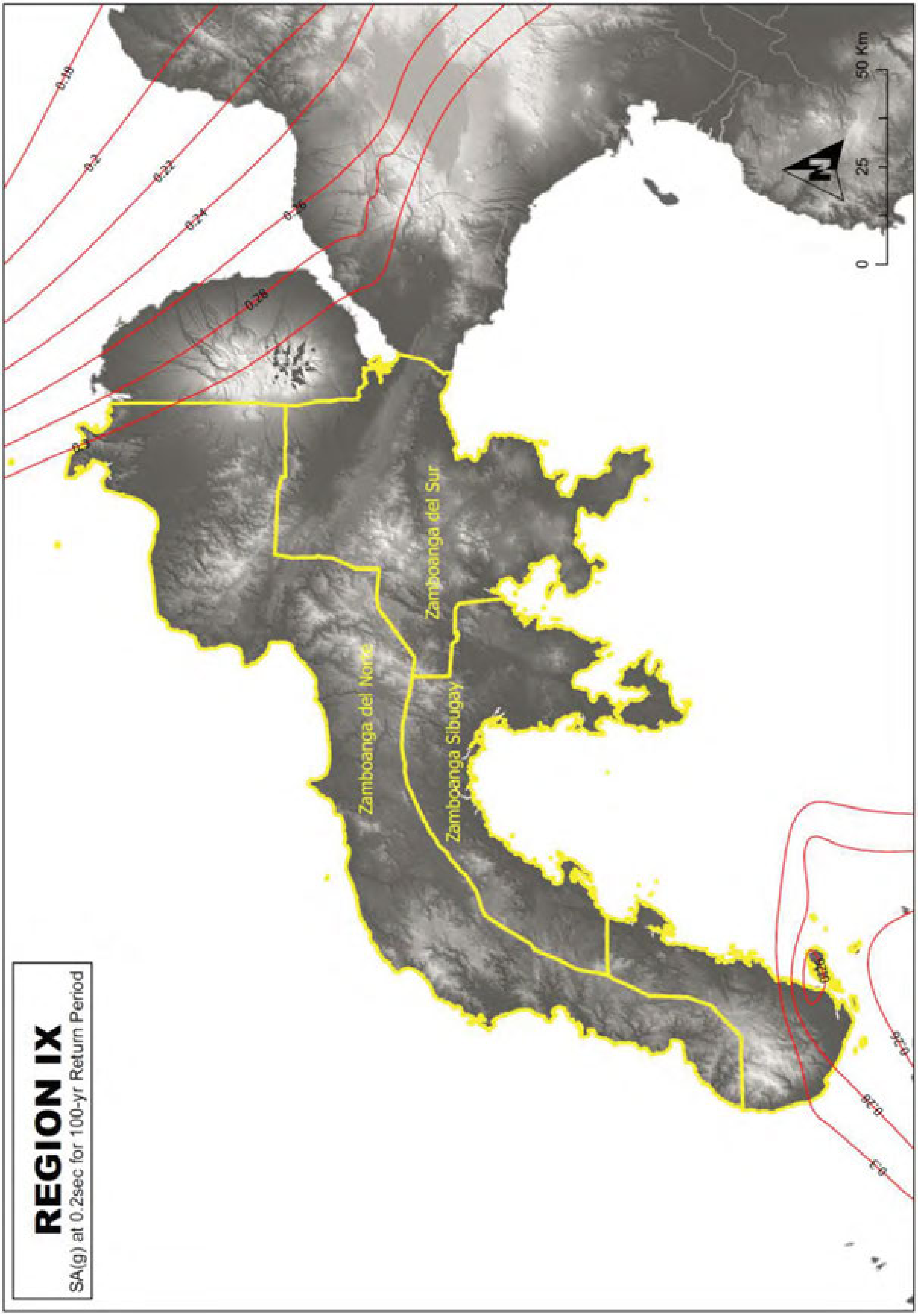


Figure 3A-31 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region IX)

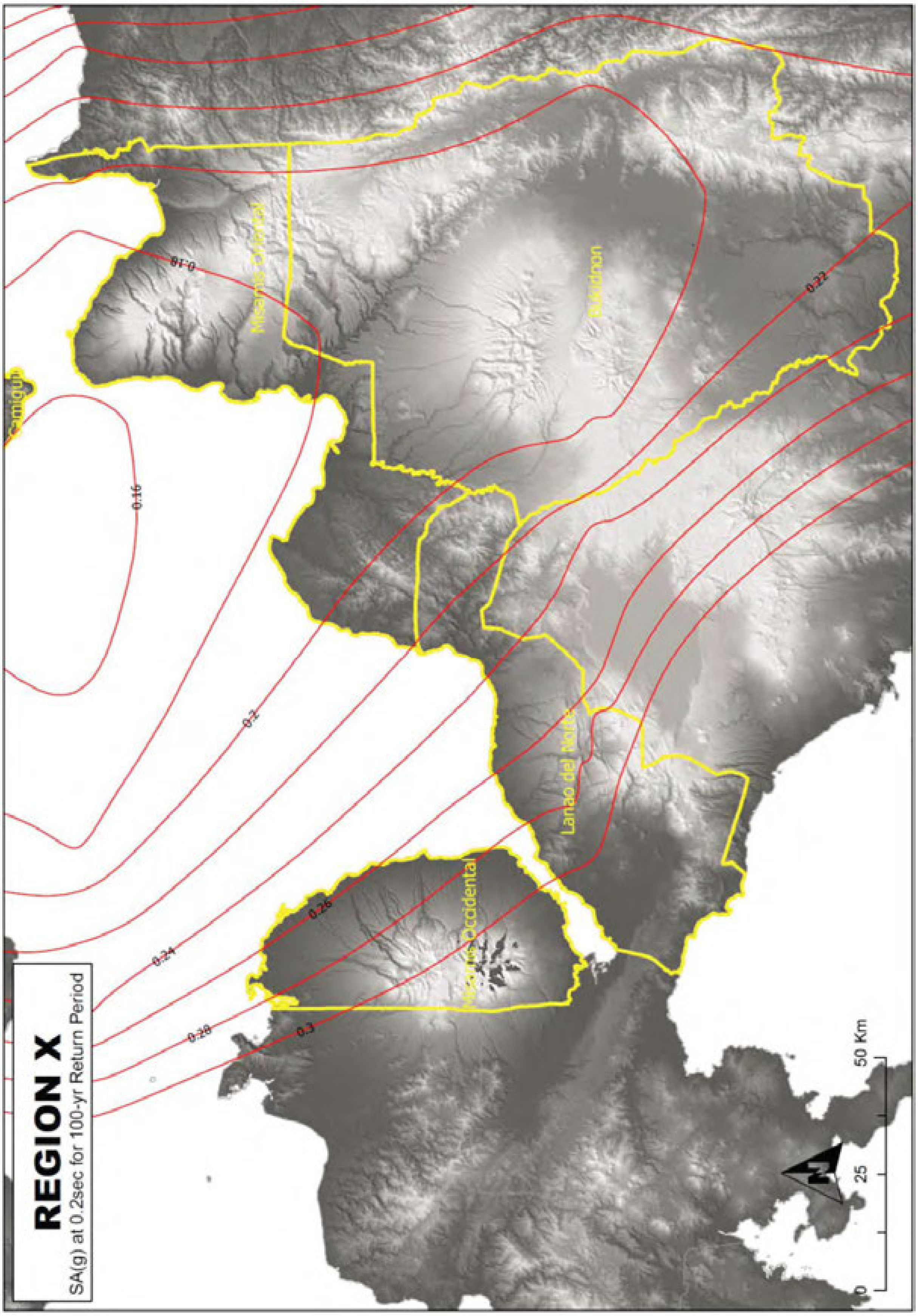


Figure 3A-32 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region X)

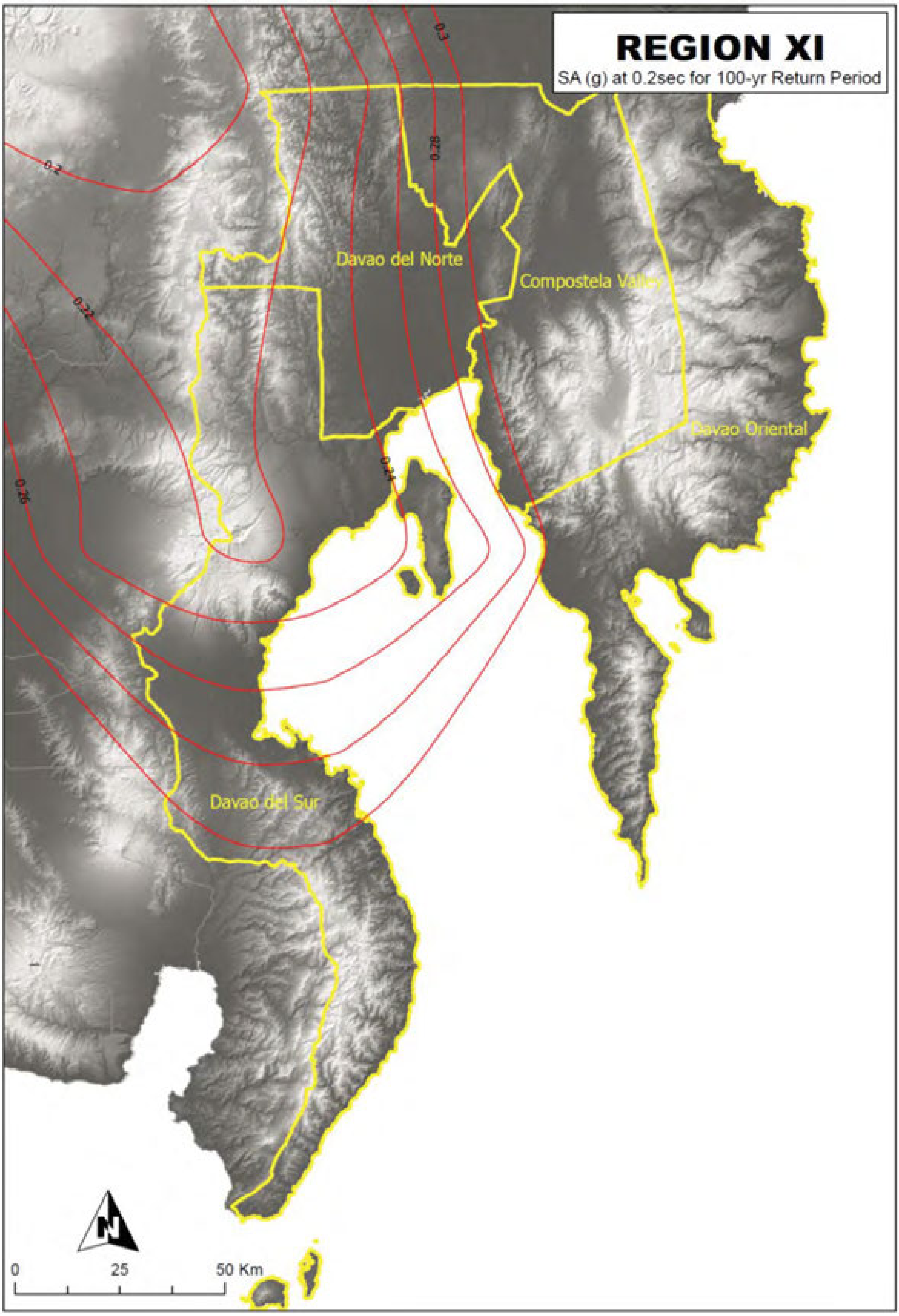


Figure 3A-33 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XI)

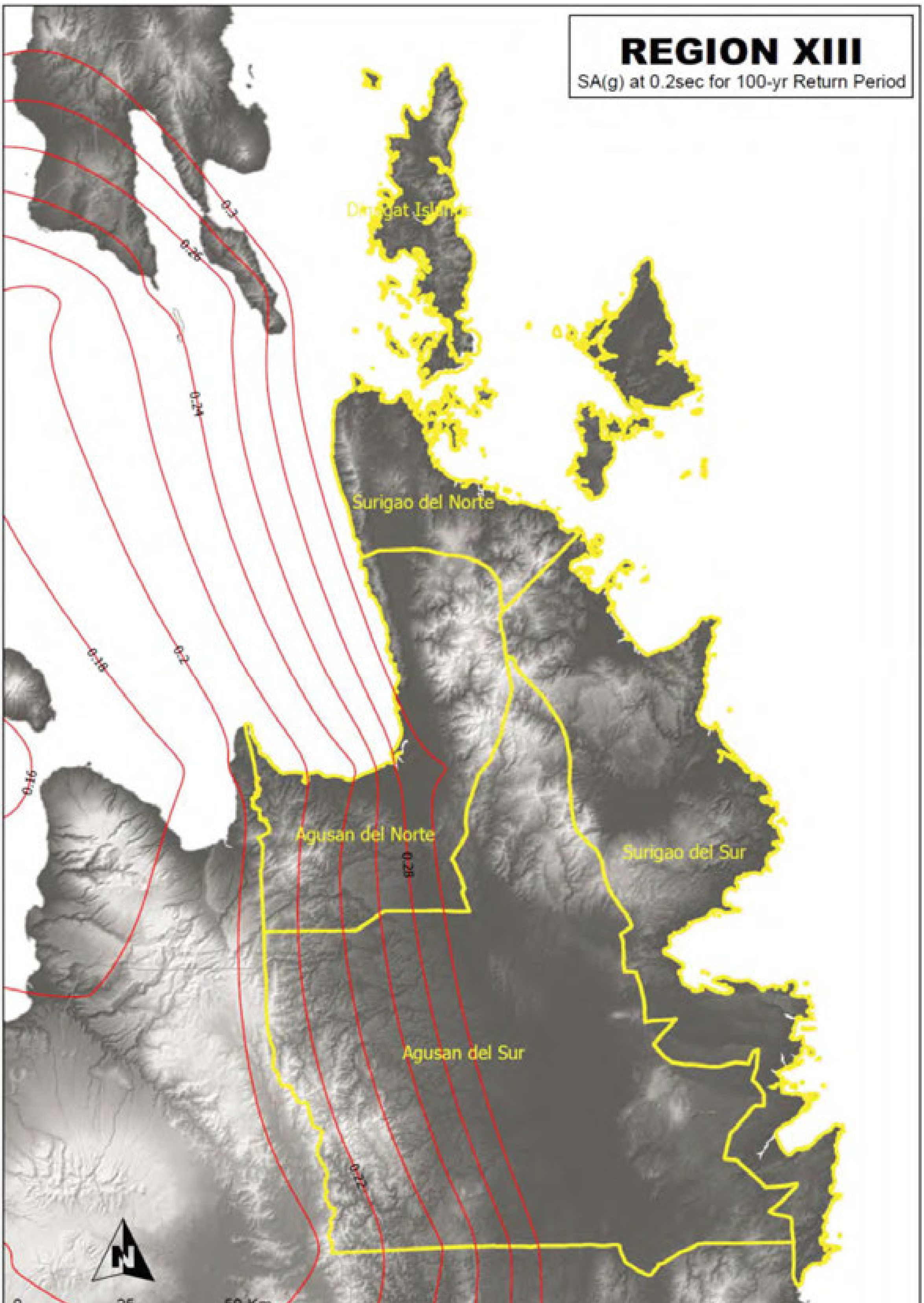


Figure 3A-34 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XIII)

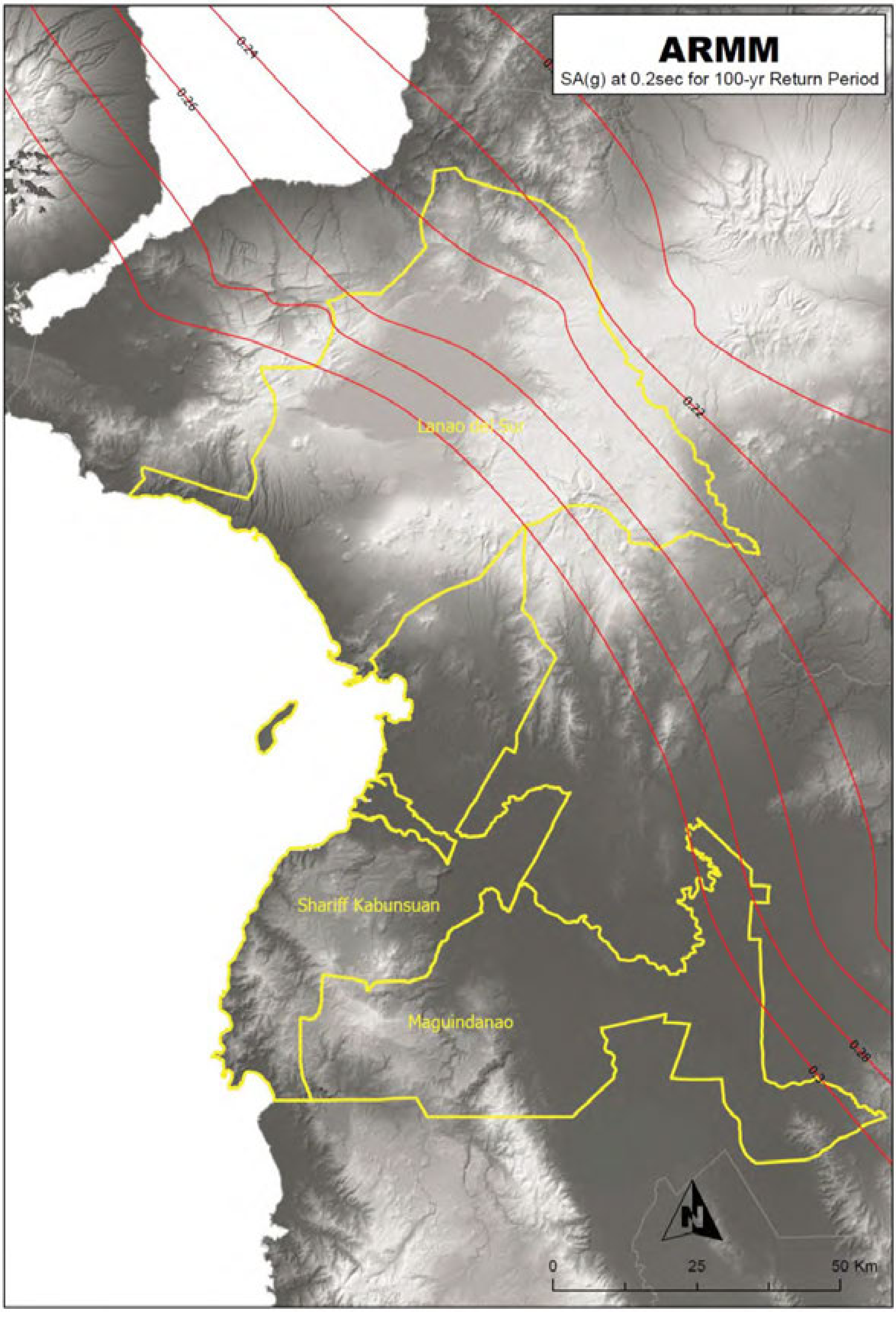


Figure 3A-35 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region ARMM)

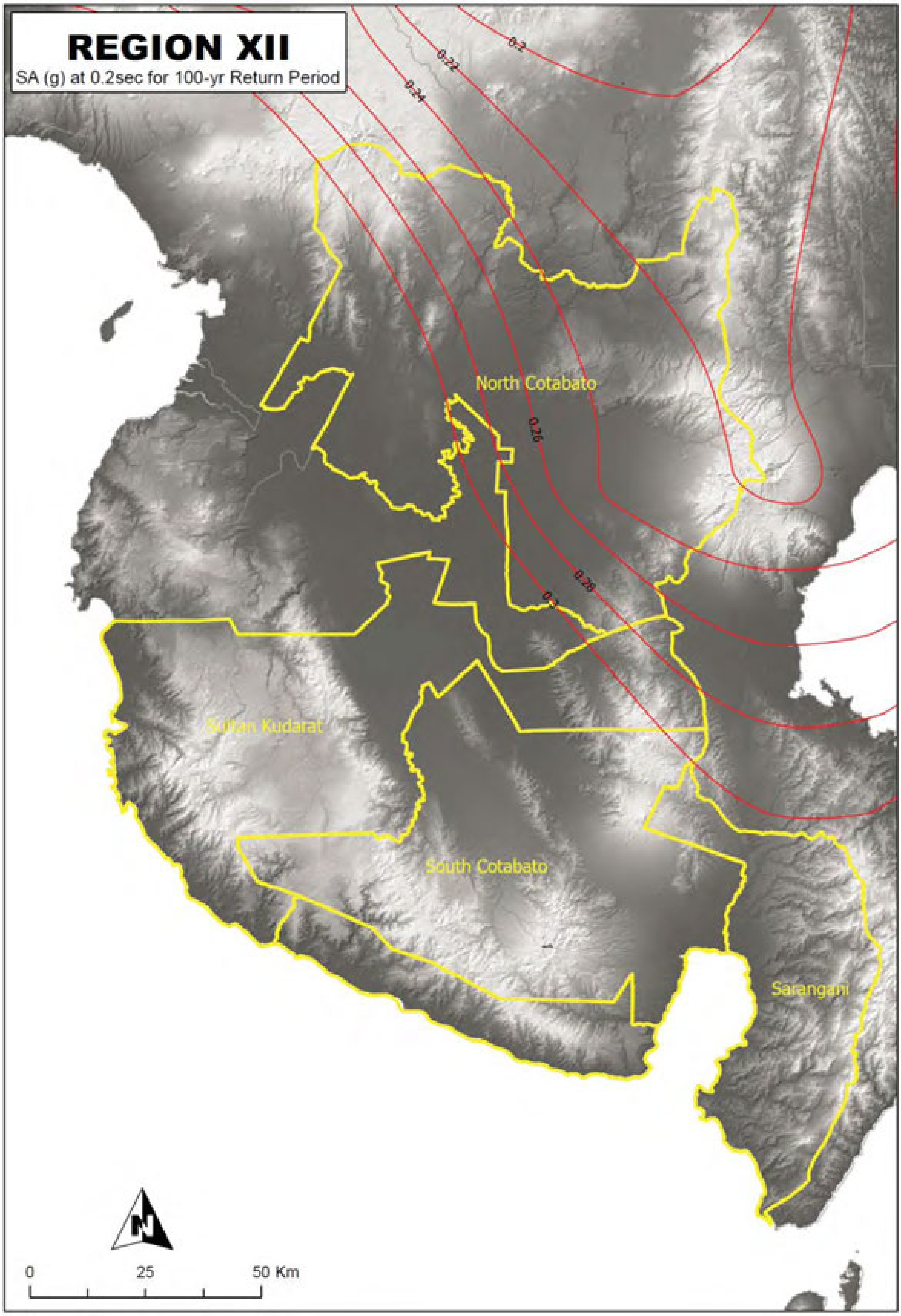


Figure 3A-36 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_S) for Level 1 Earthquake Ground Motion (Region XII)

HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT 1.0-SEC PERIOD

(100-YEAR RETURN PERIOD)

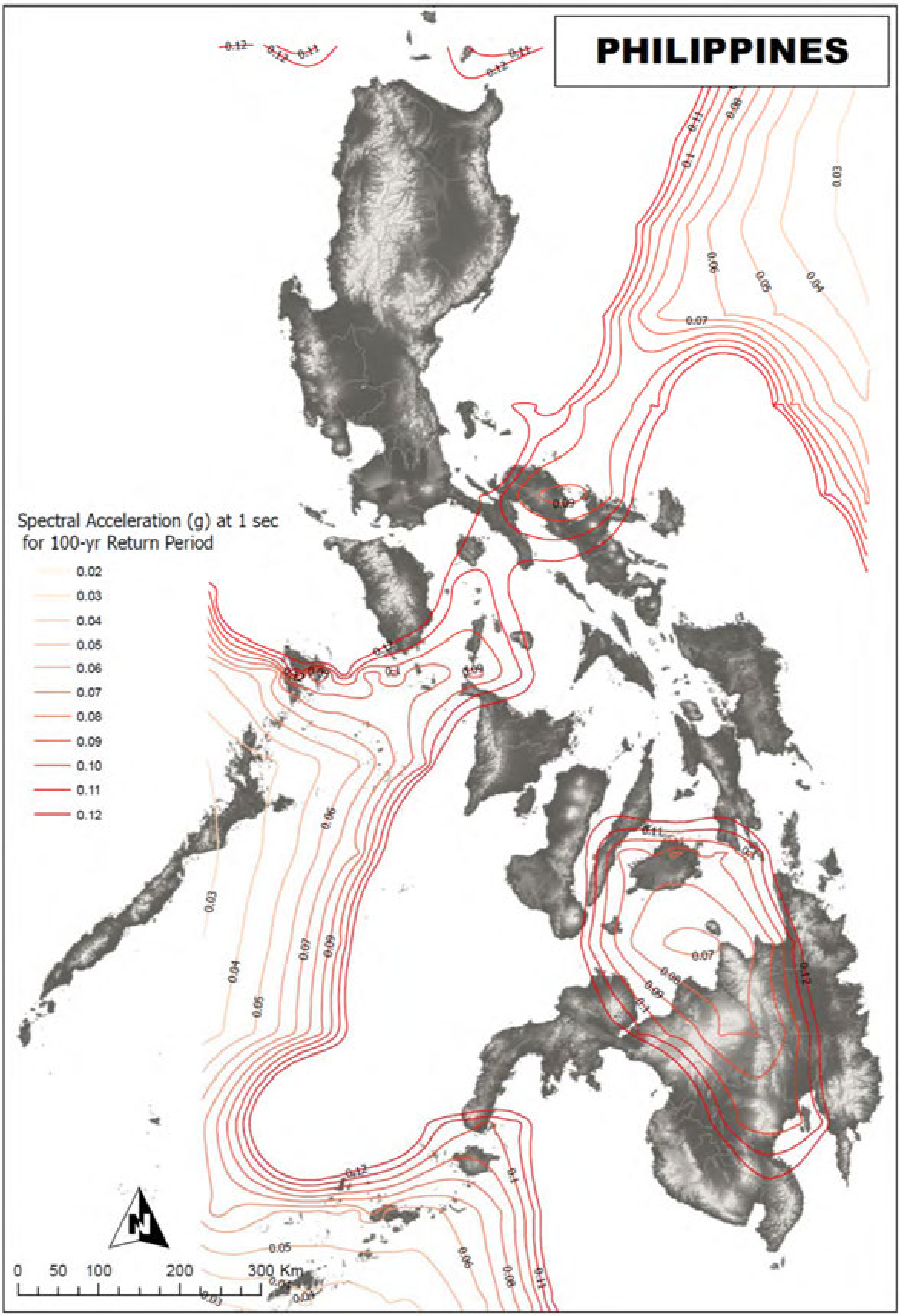


Figure 3A-37 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Entire Country)

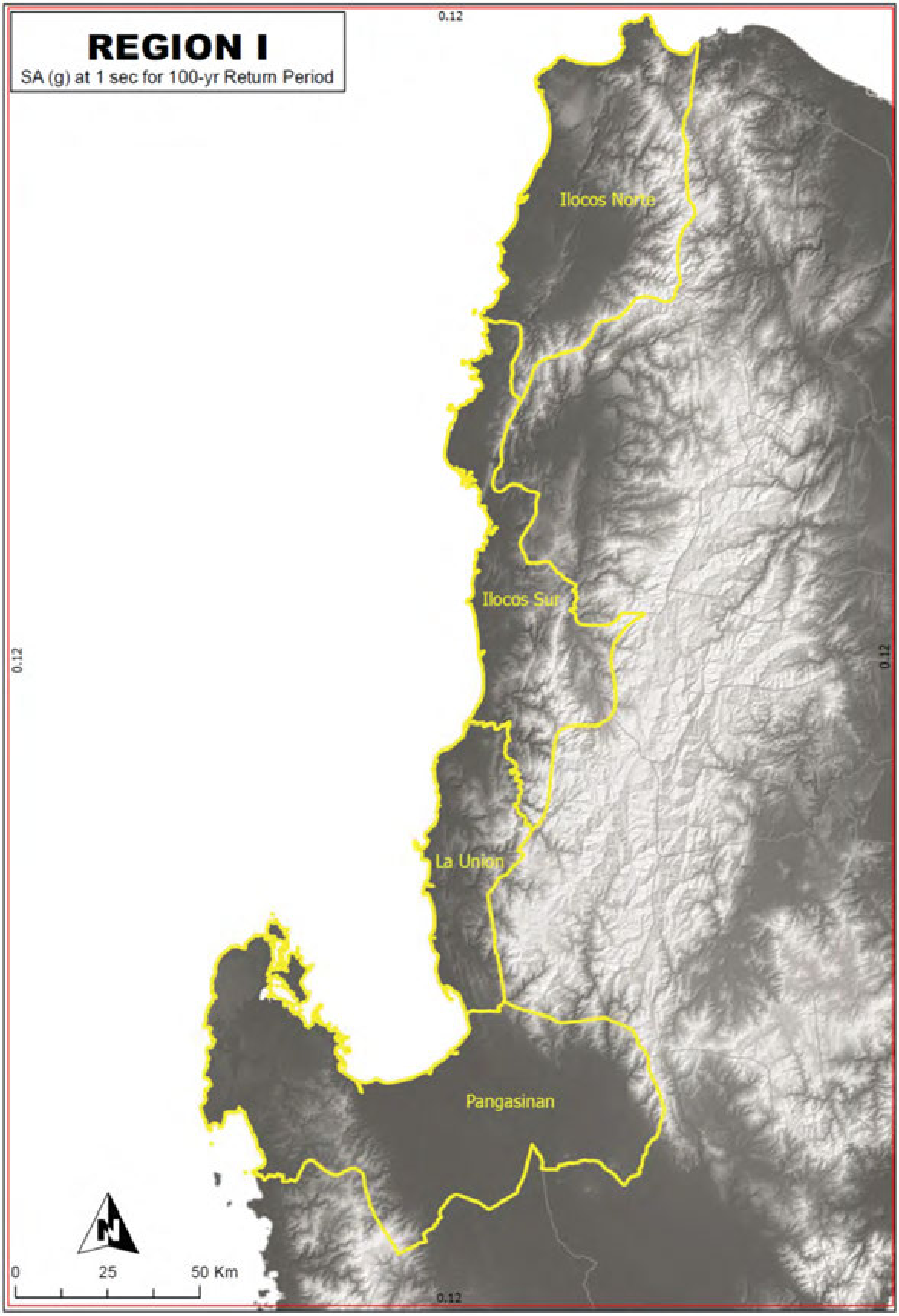


Figure 3A-38 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) Level 1 Earthquake Ground Motion (Region I)

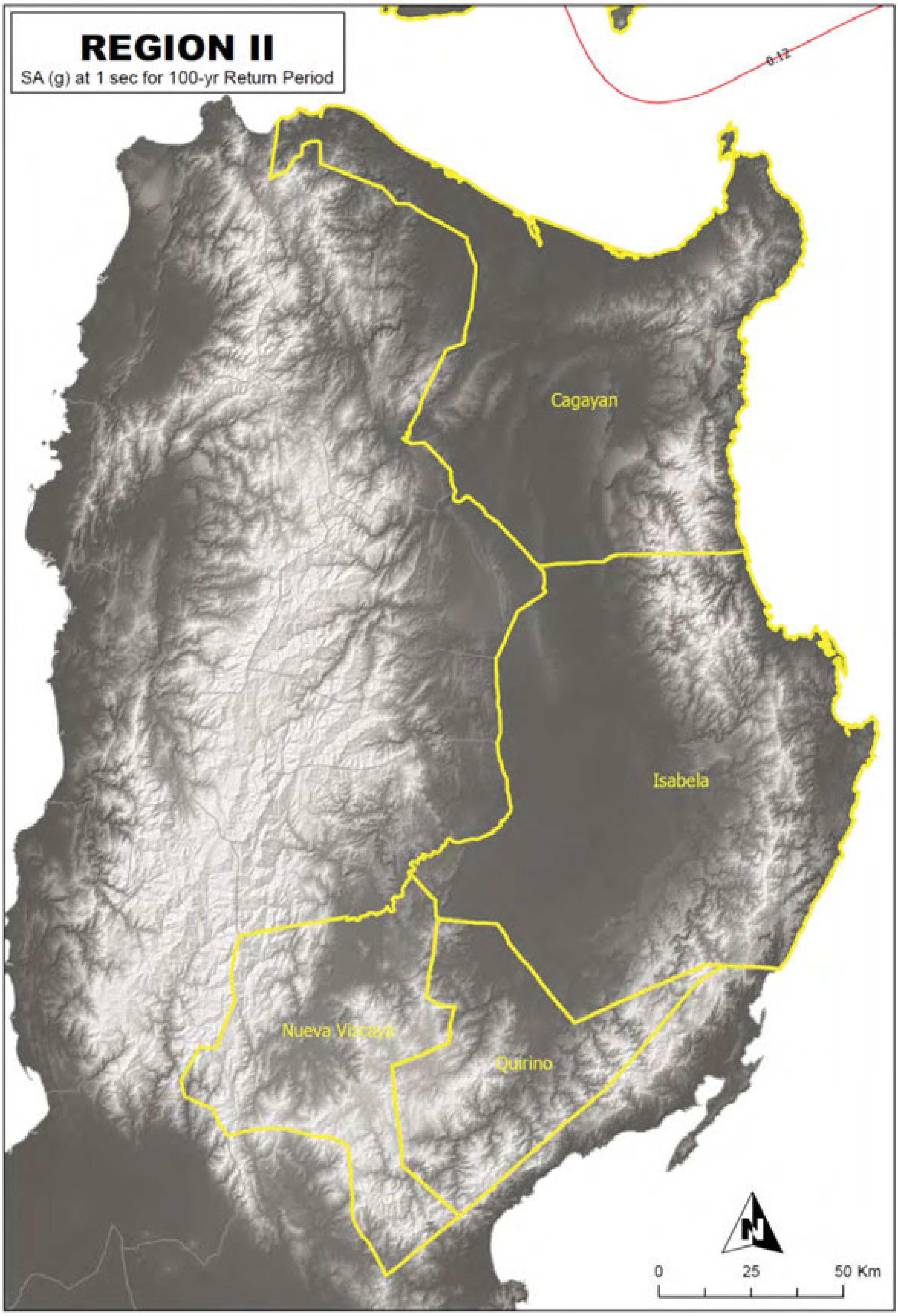


Figure 3A-39 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region II)

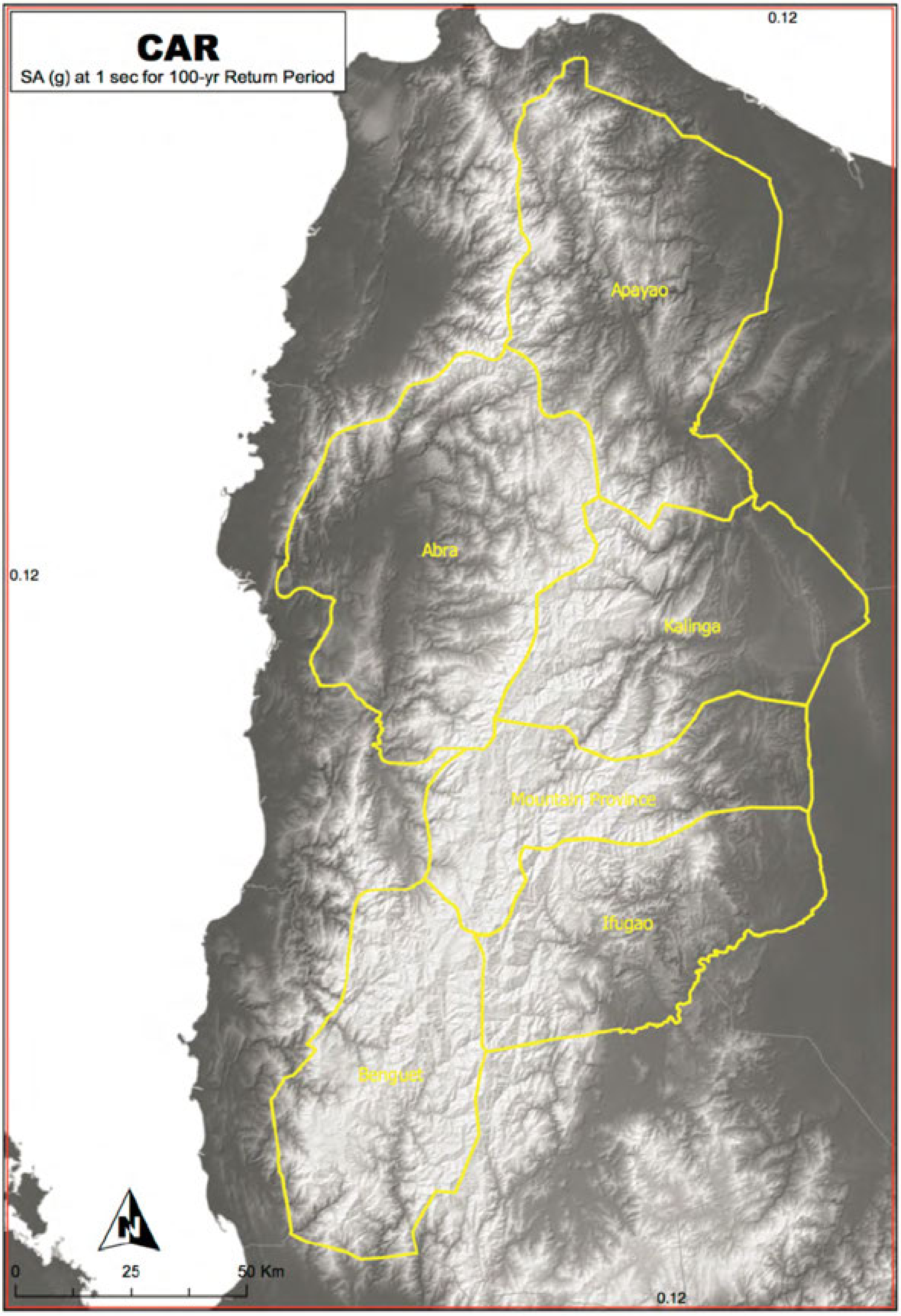


Figure 3A-40 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (CAR)

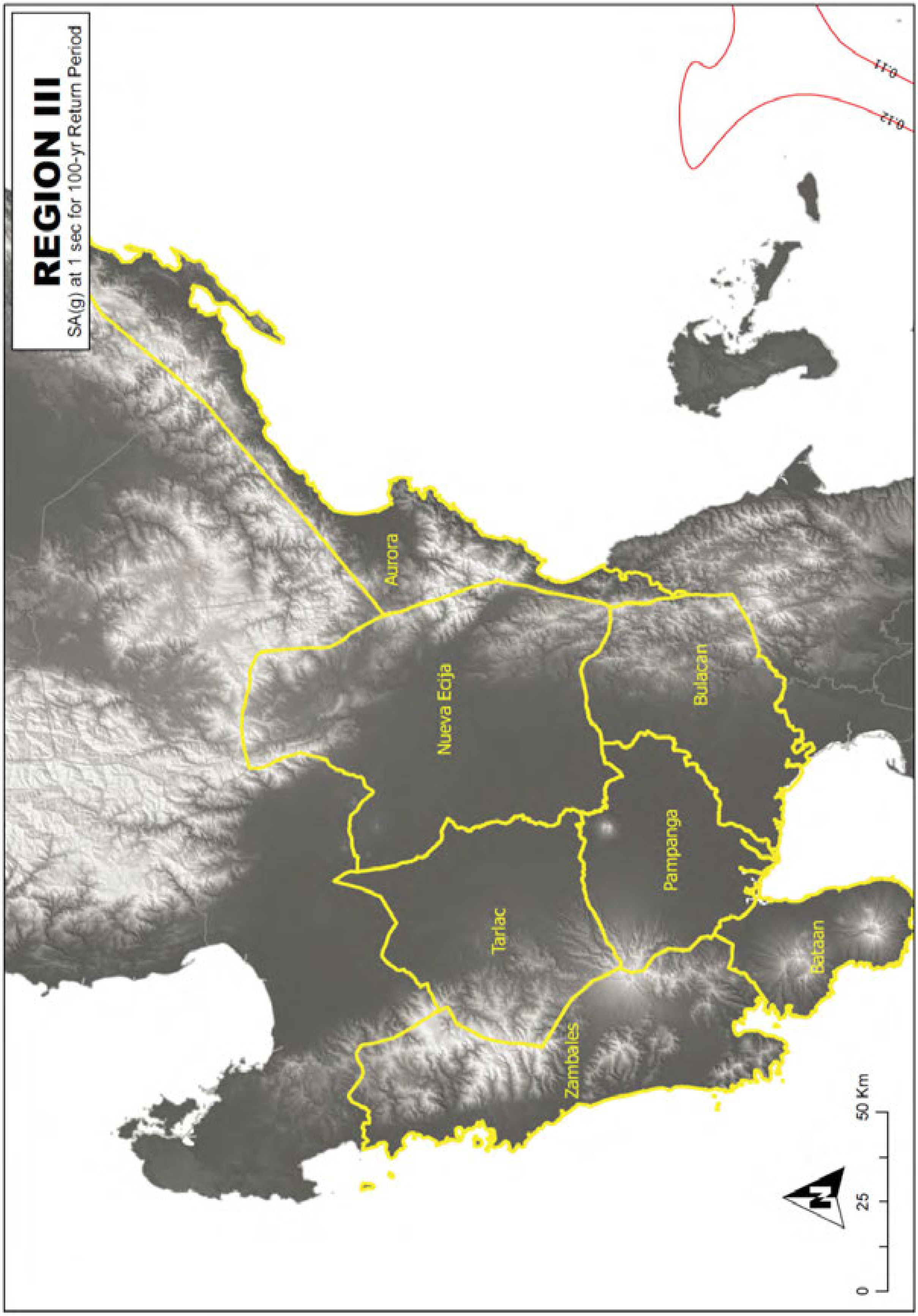


Figure 3A-41 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region III)

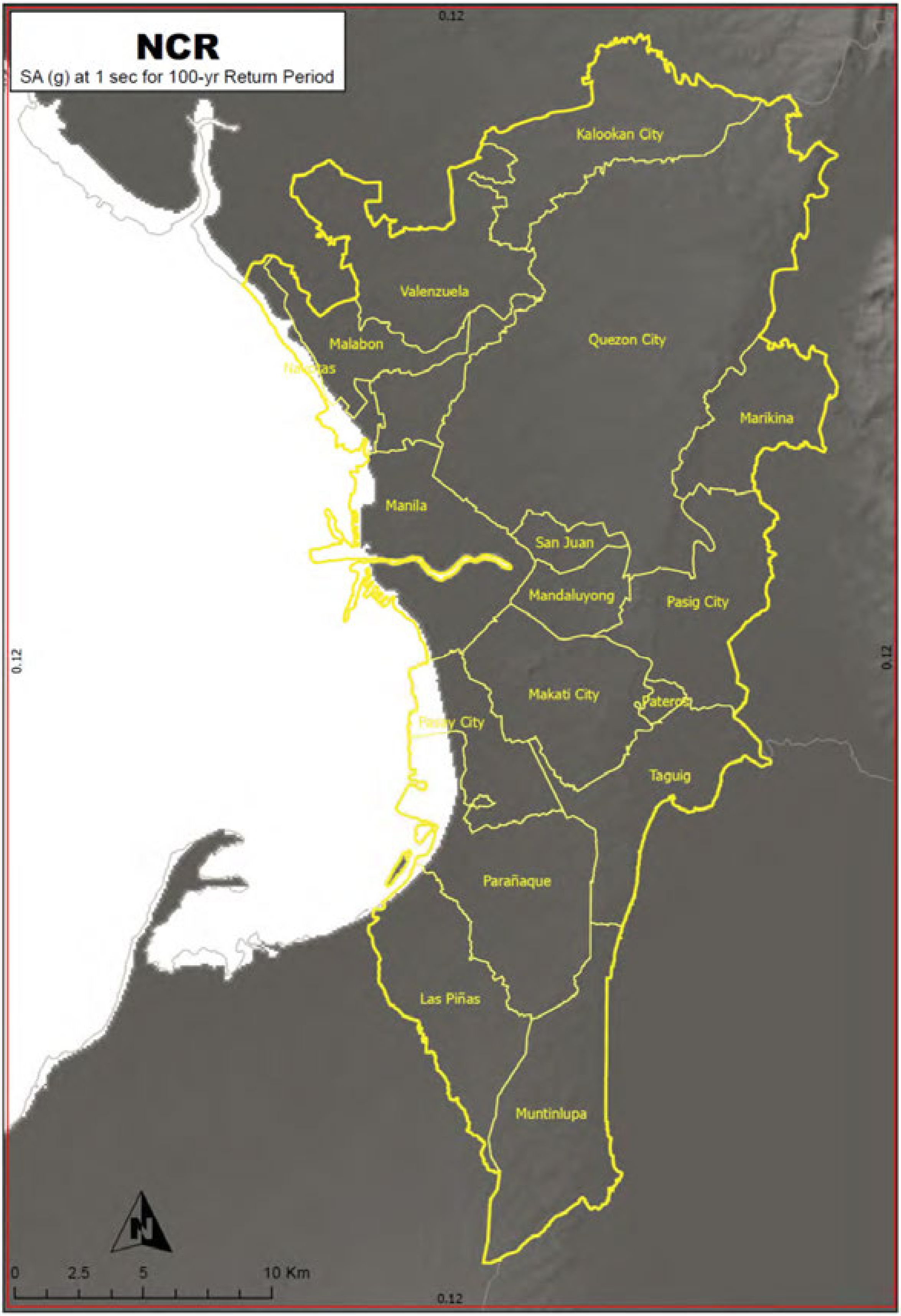


Figure 3A-42 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (NCR)

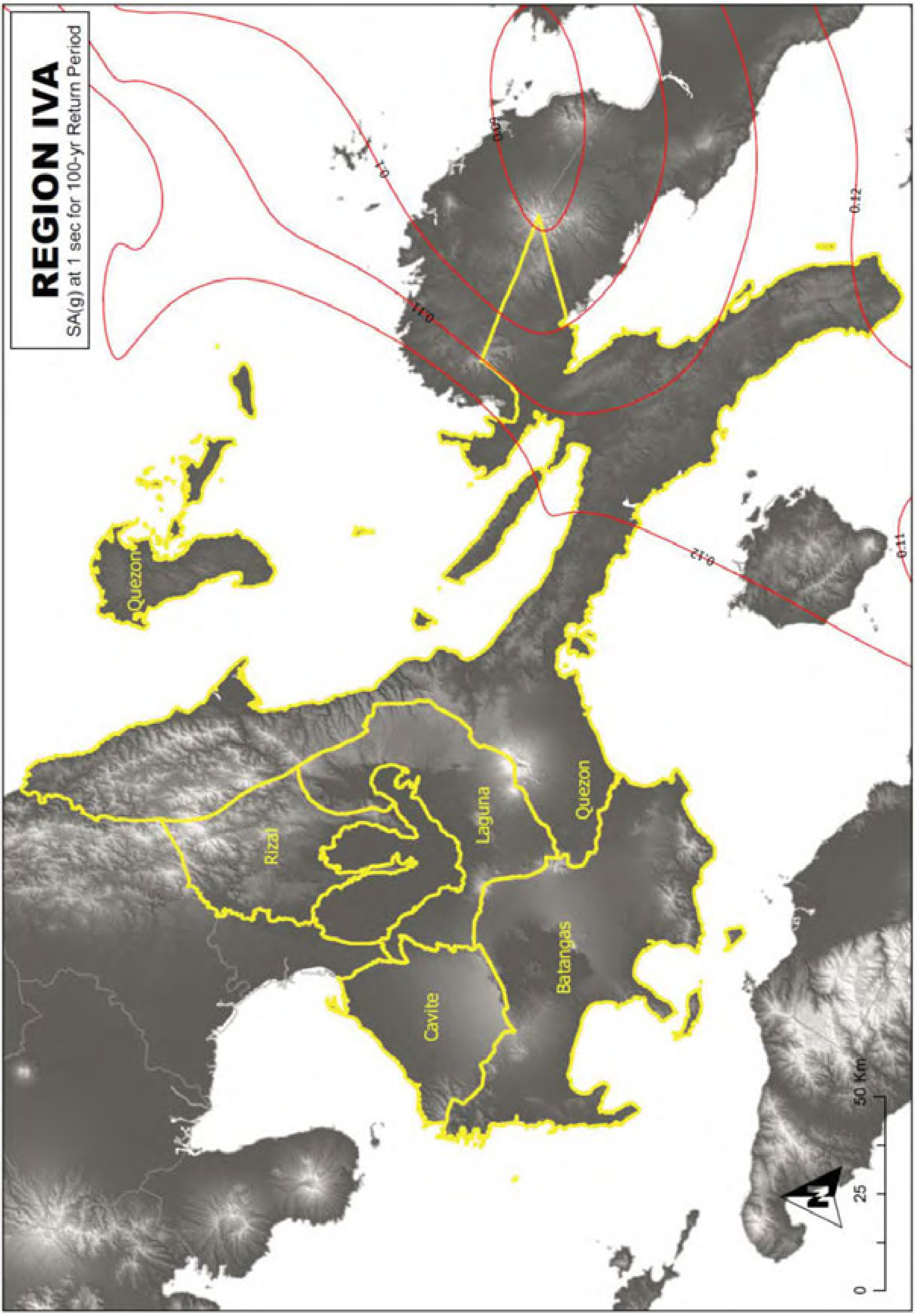


Figure 3A-43 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IV-A)

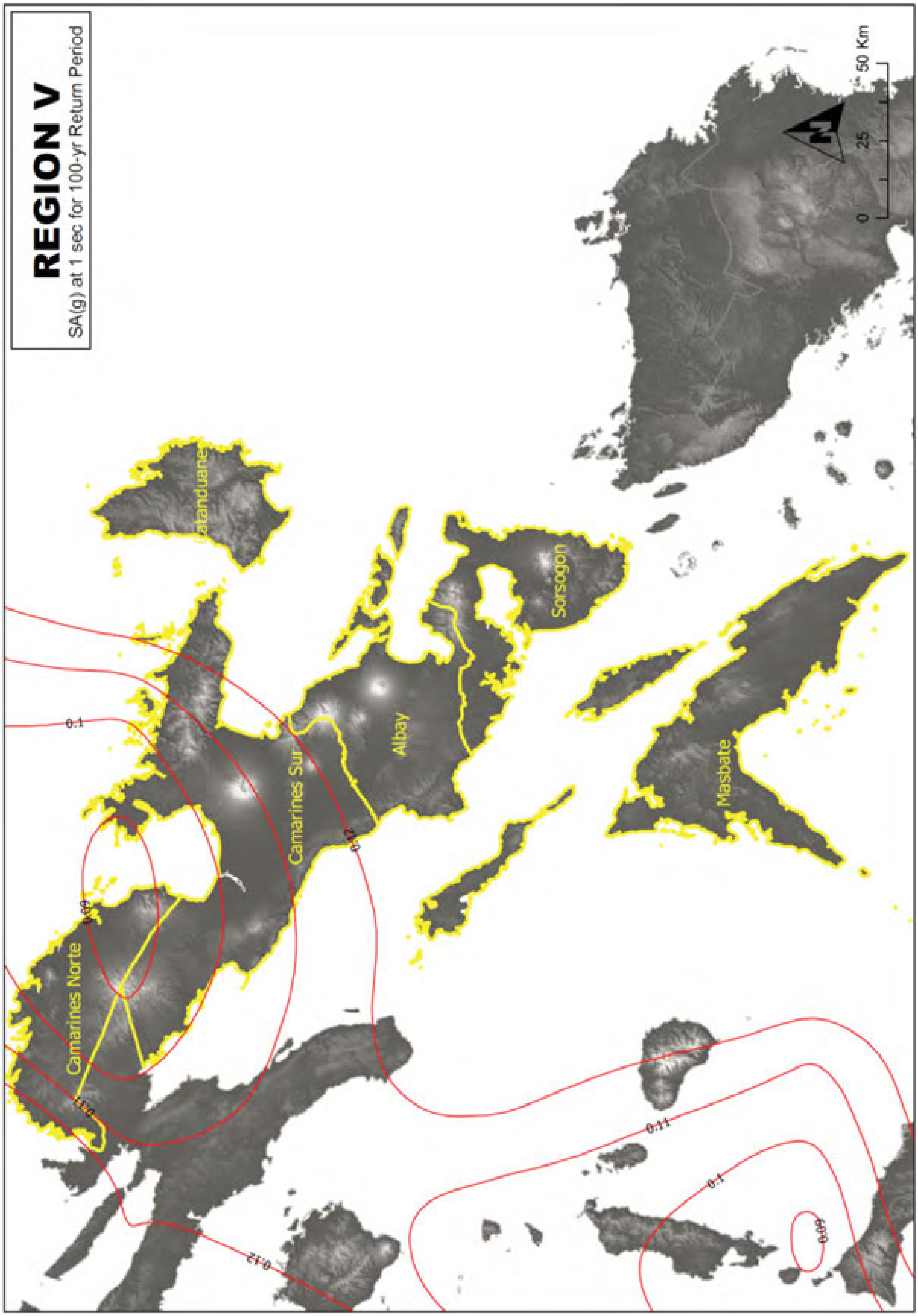


Figure 3A-44 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region V)

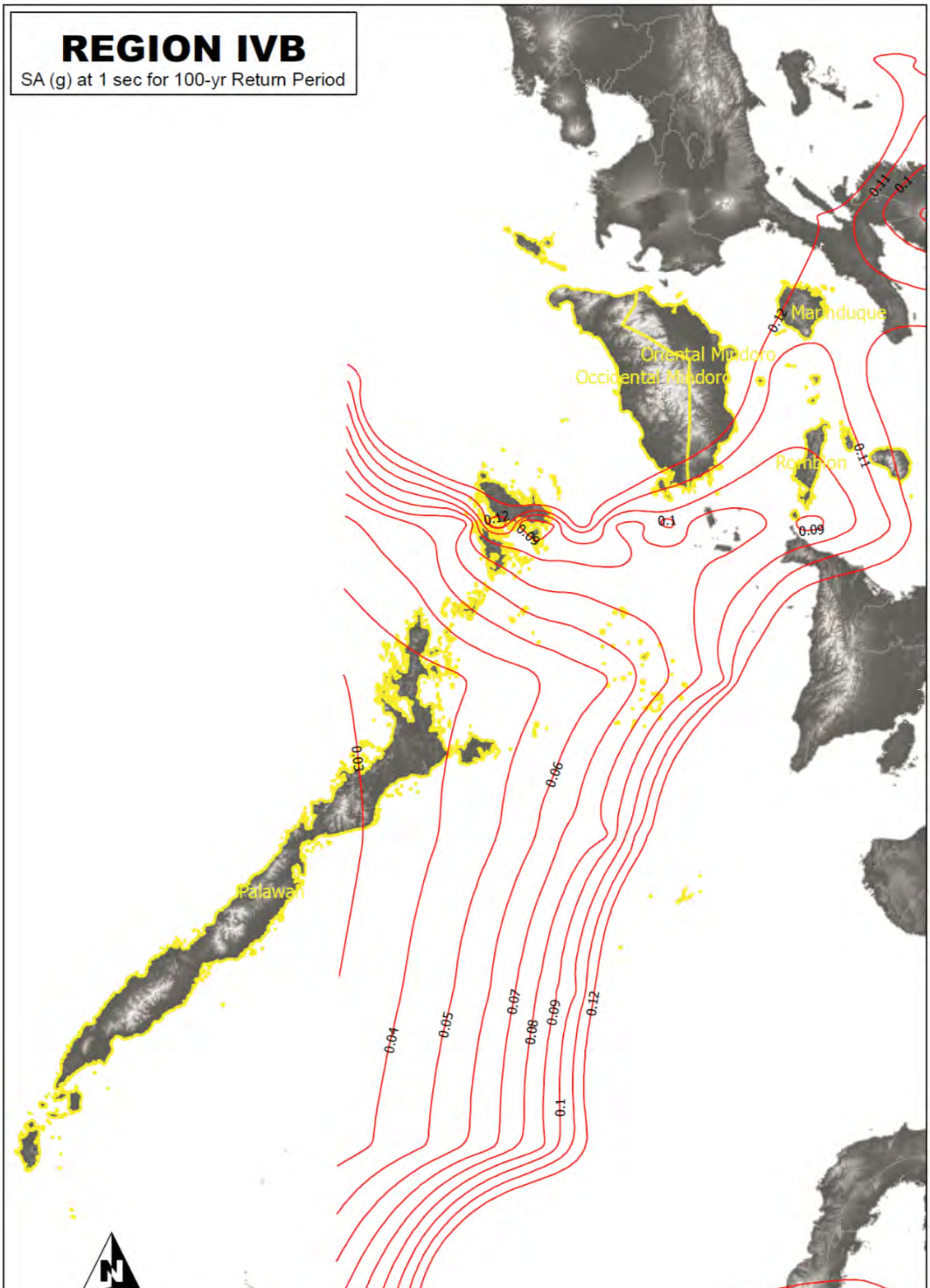


Figure 3A-45 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IV-B)

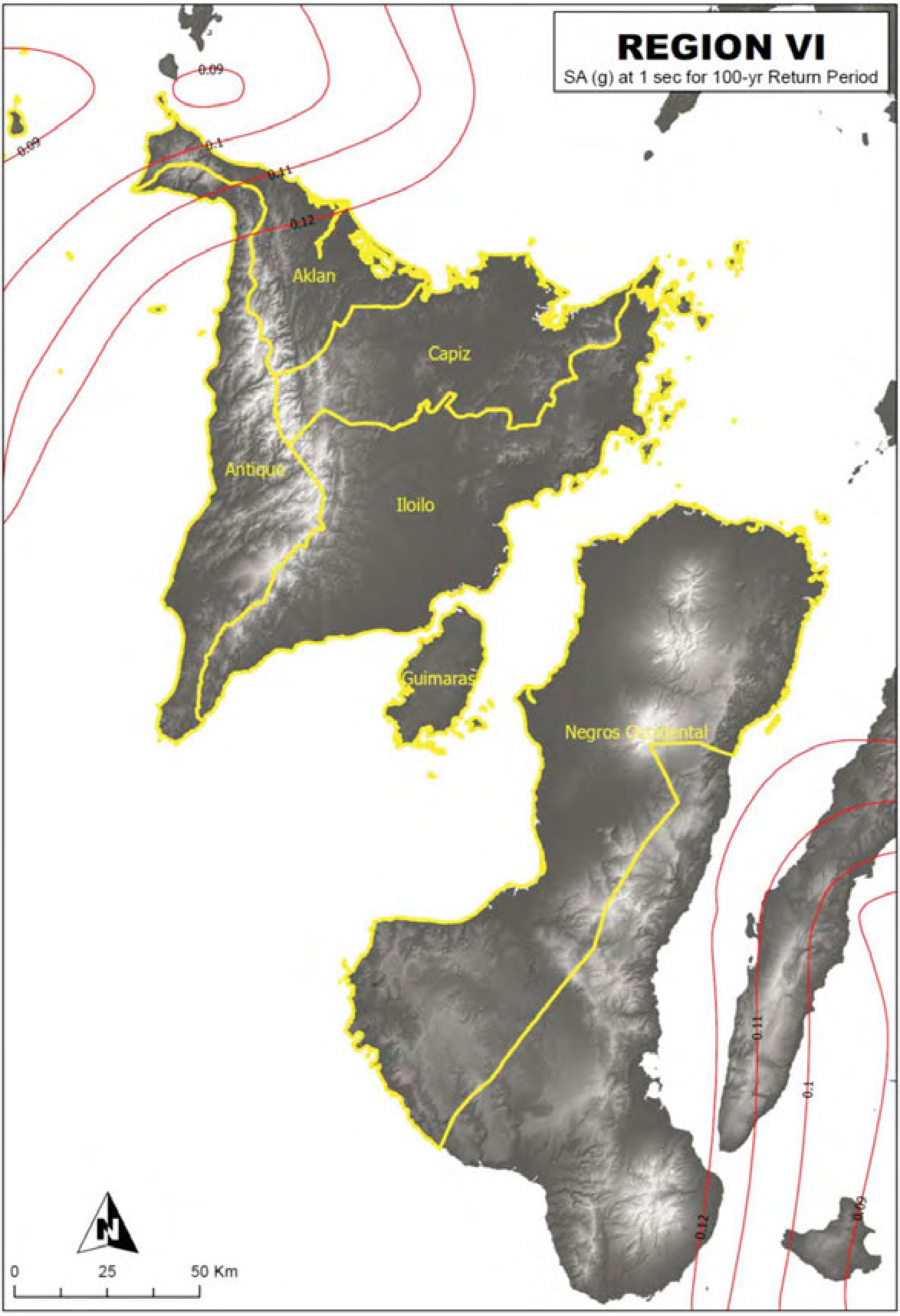


Figure 3A-46 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VI)

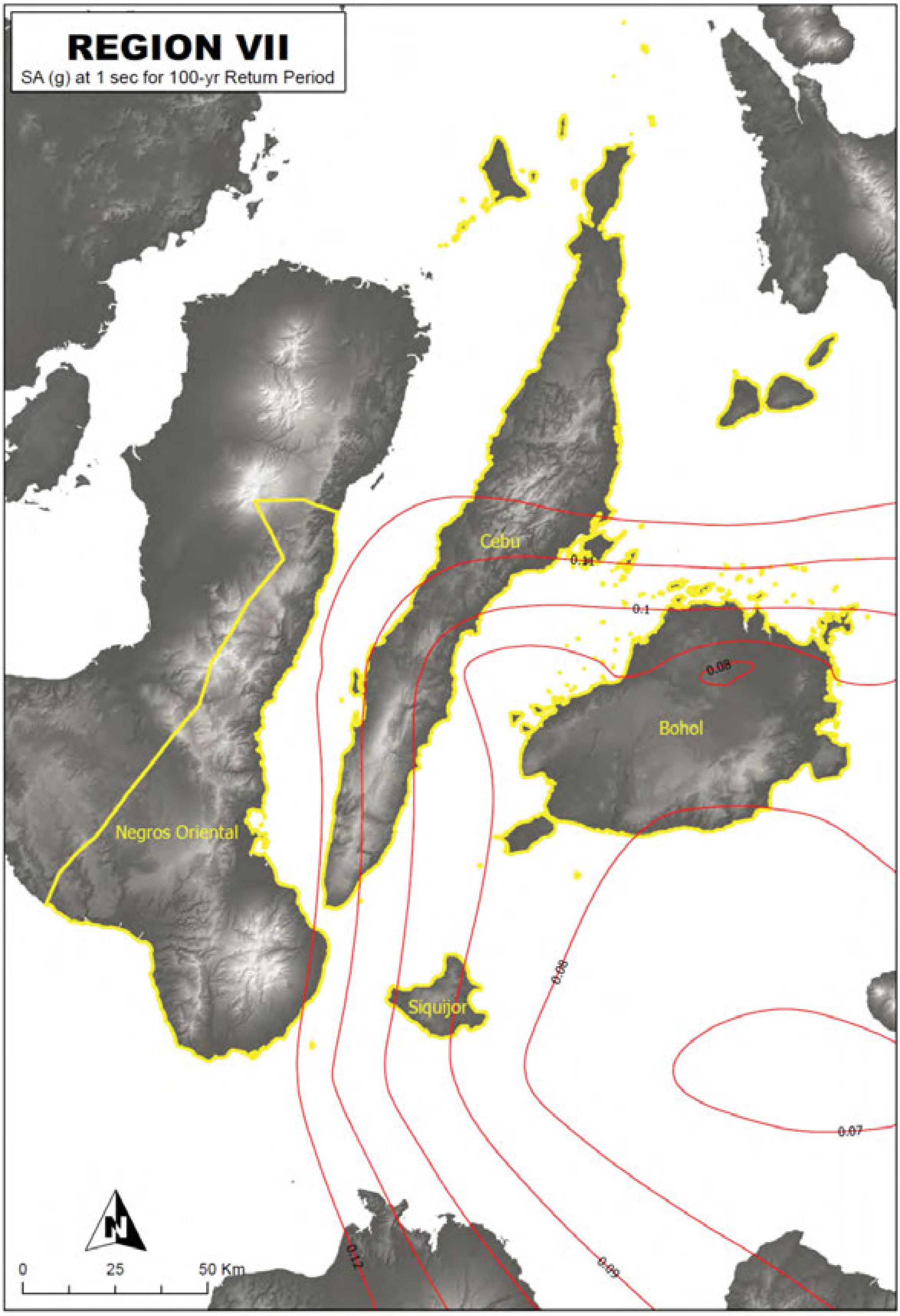


Figure 3A-47 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VII)

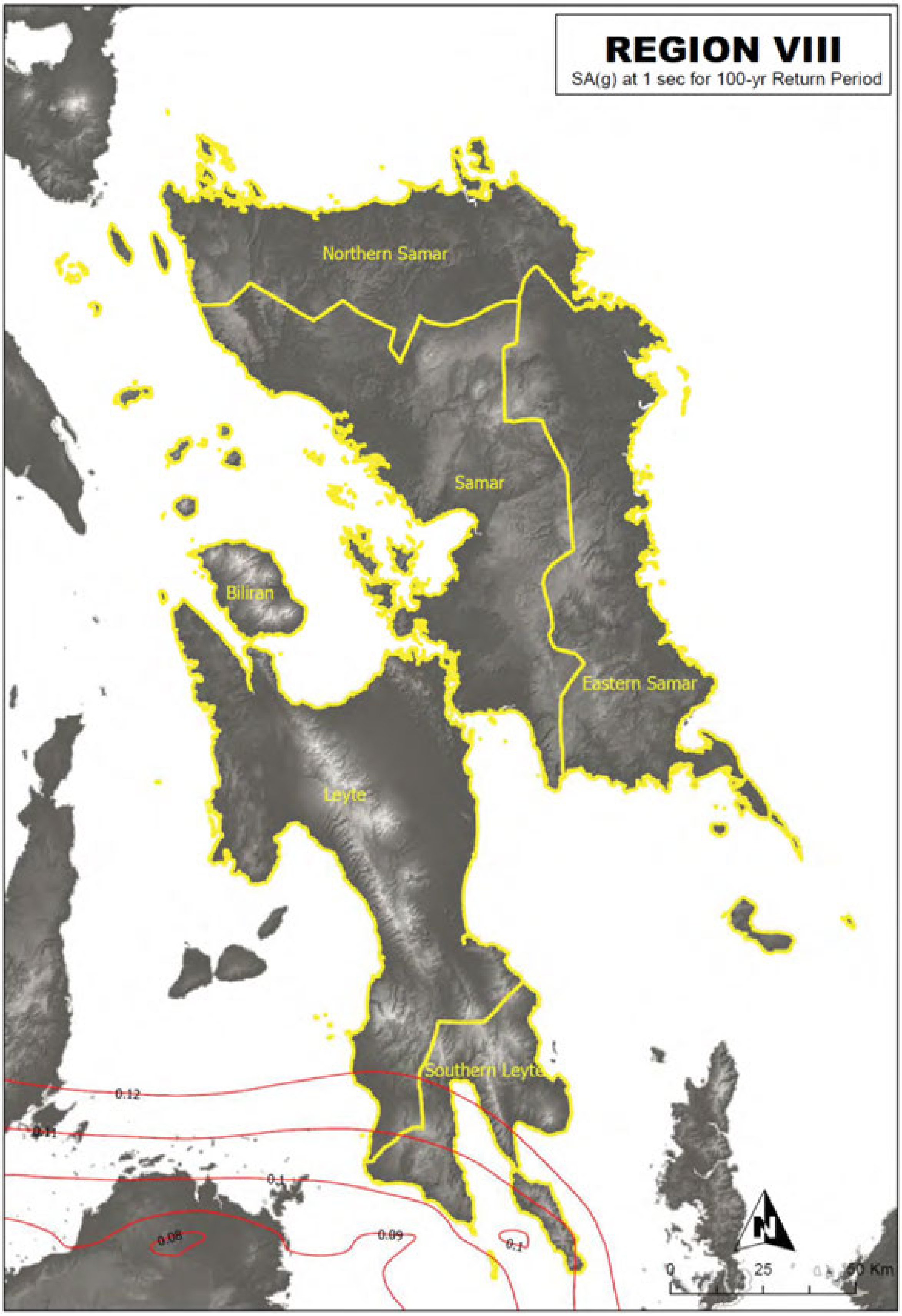


Figure 3A-48 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region VIII)

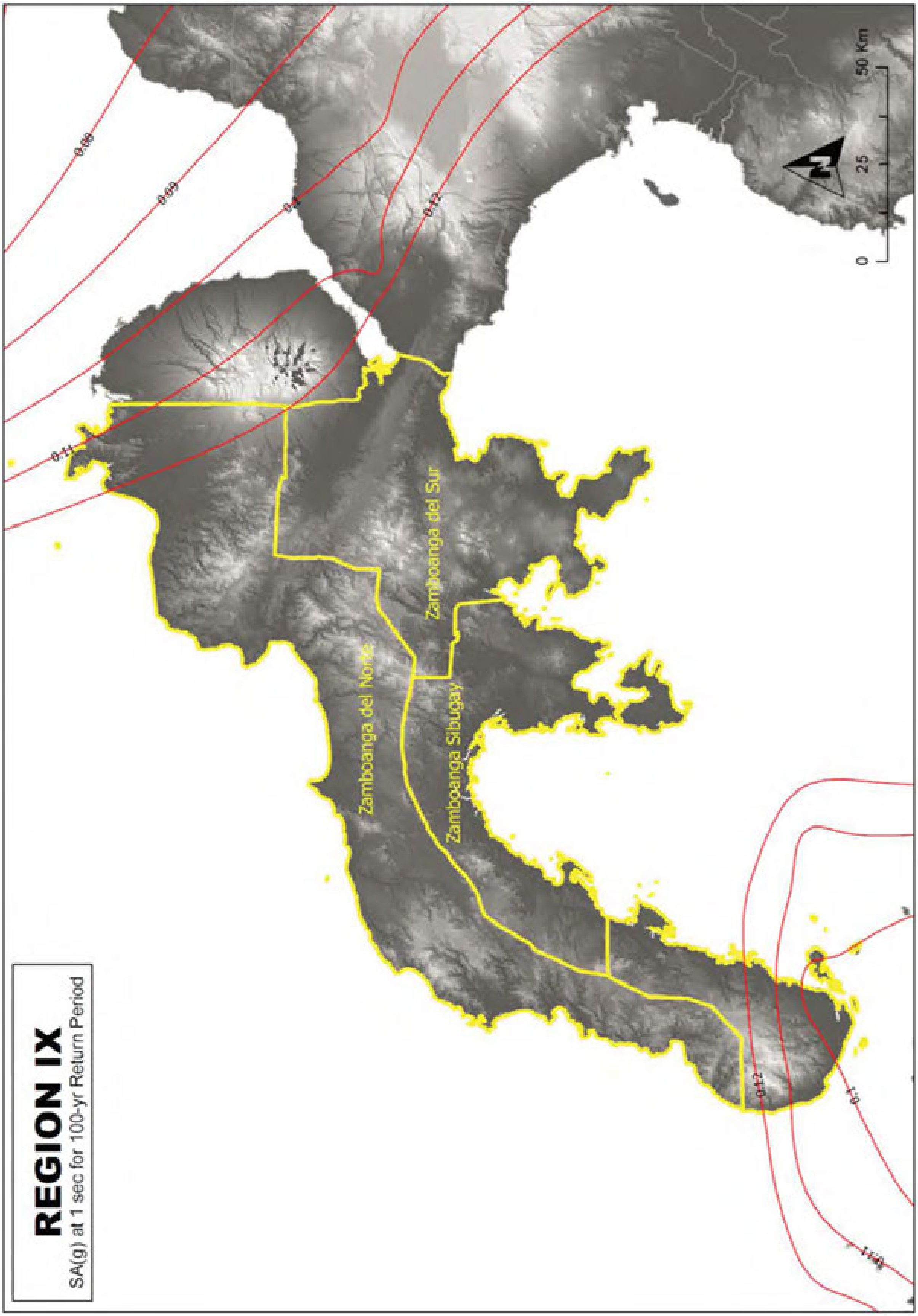


Figure 3A-49 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region IX)

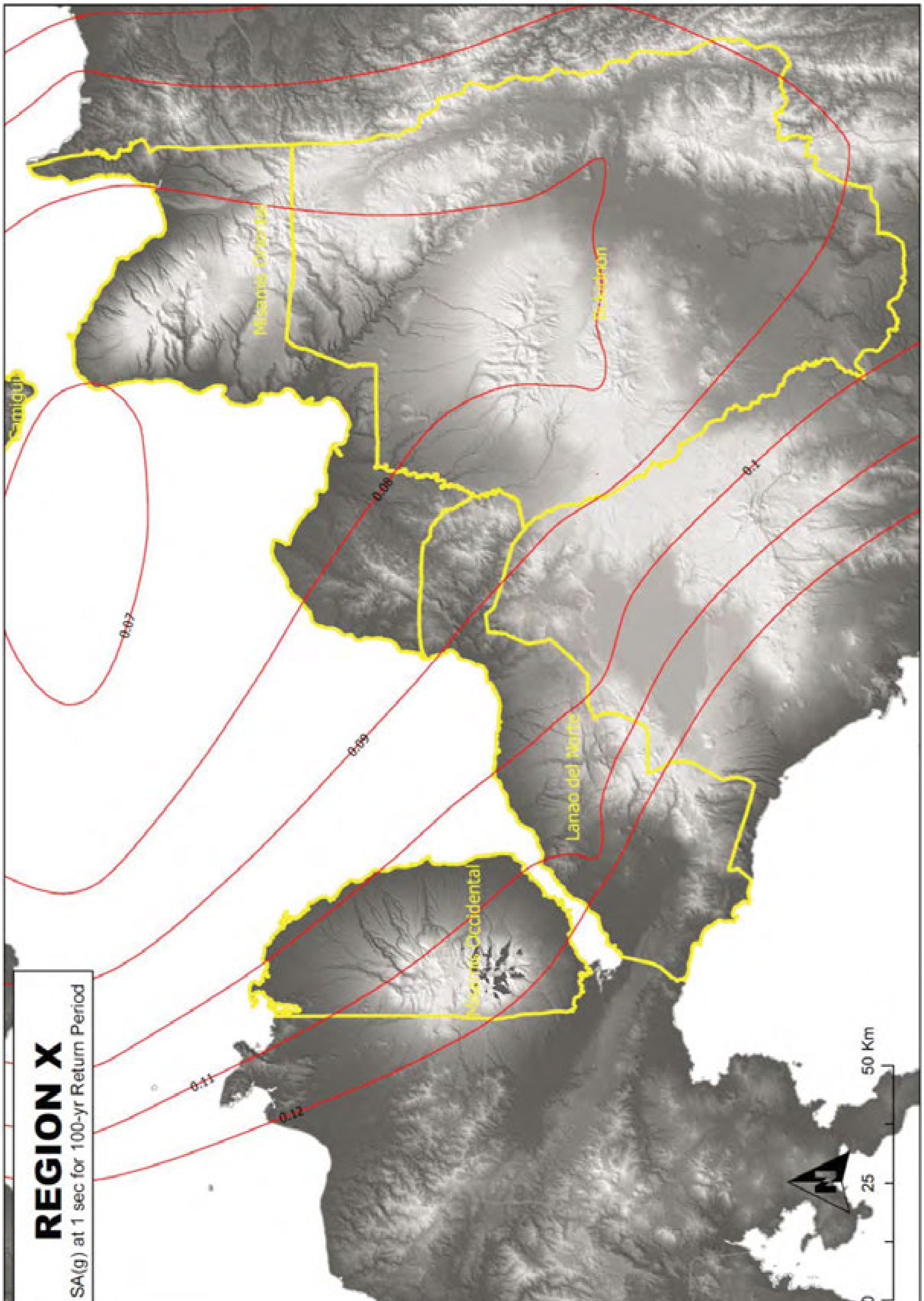


Figure 3A-50 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region X)

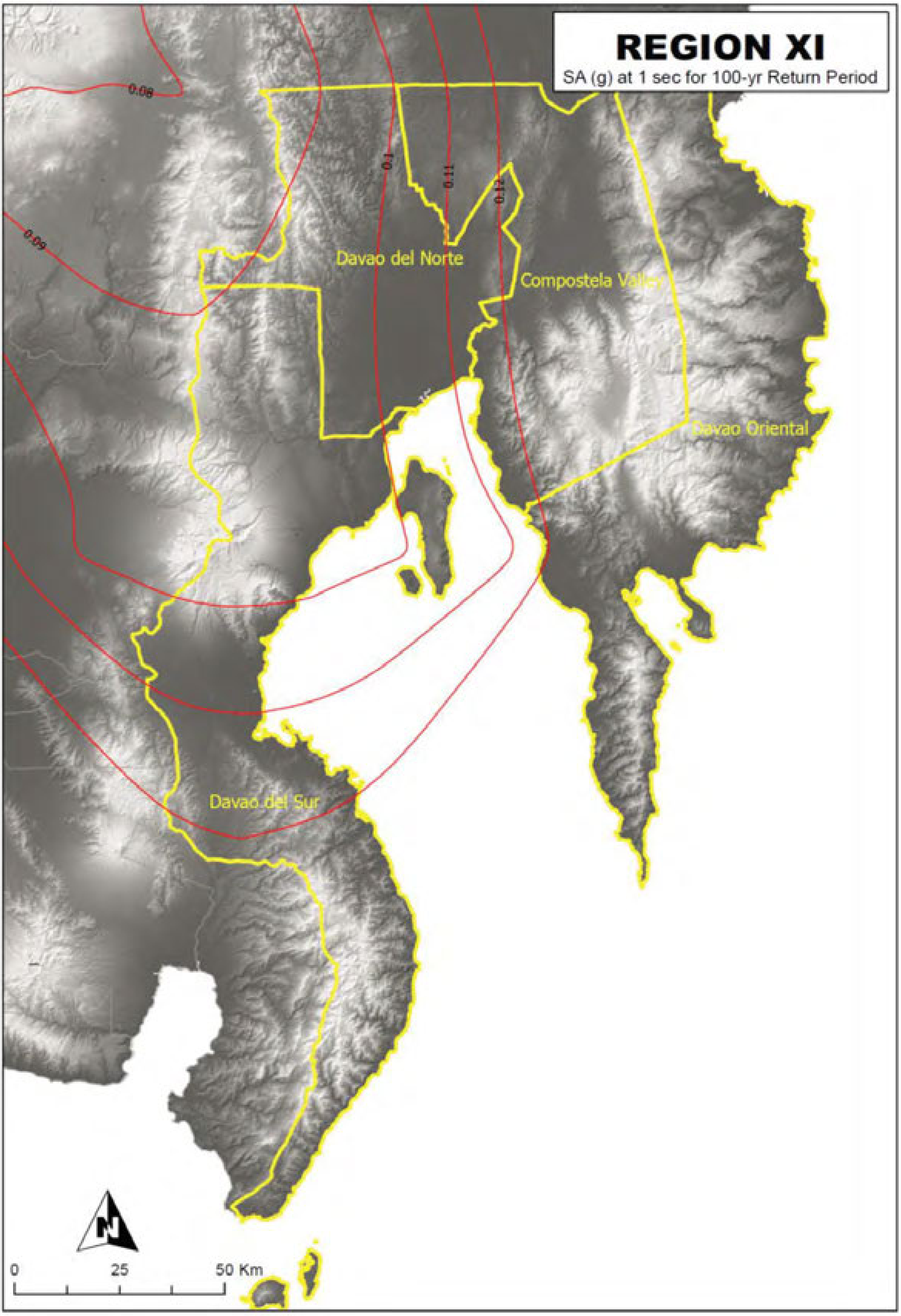


Figure 3A-51 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XI)

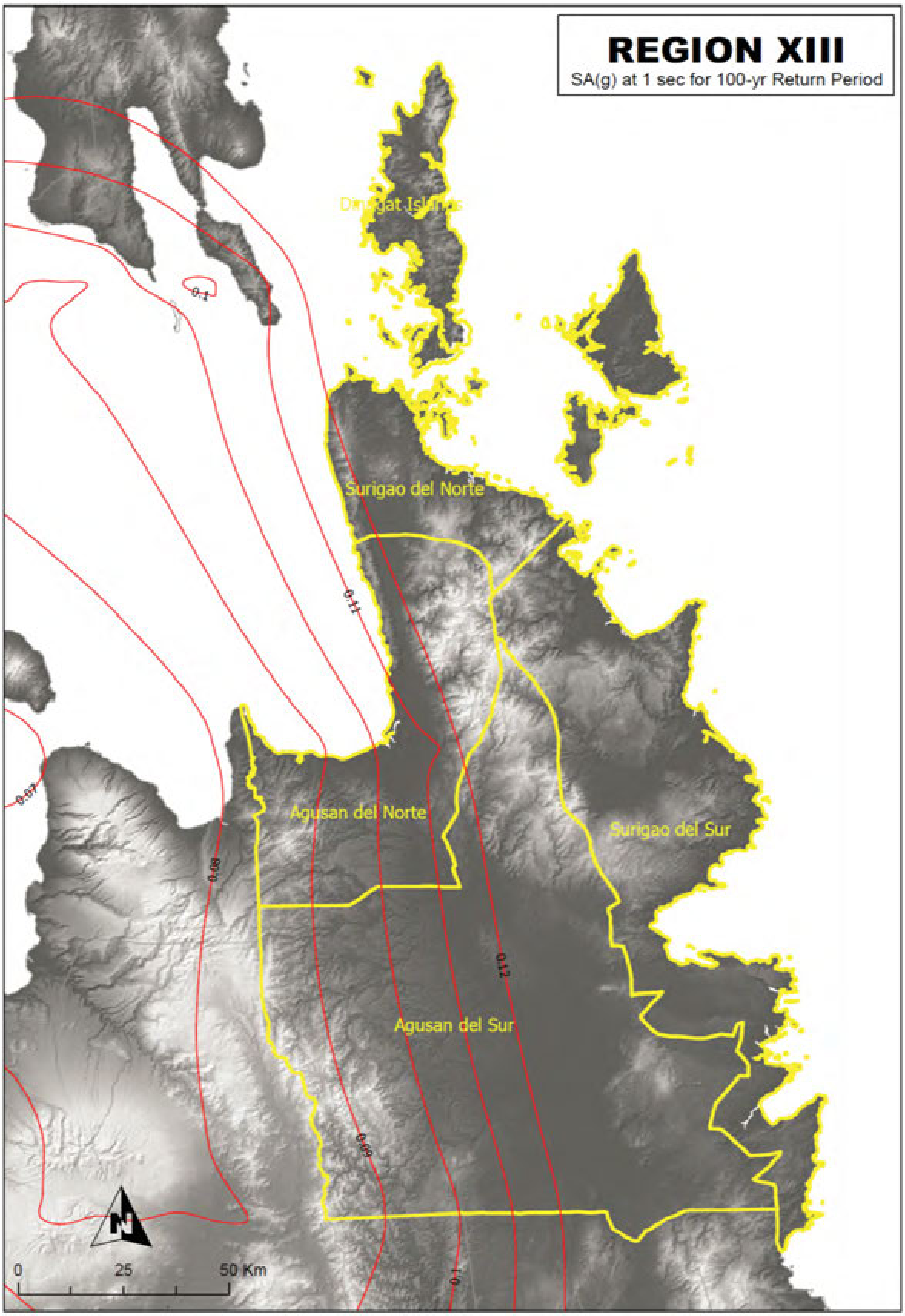


Figure 3A-52 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XIII)

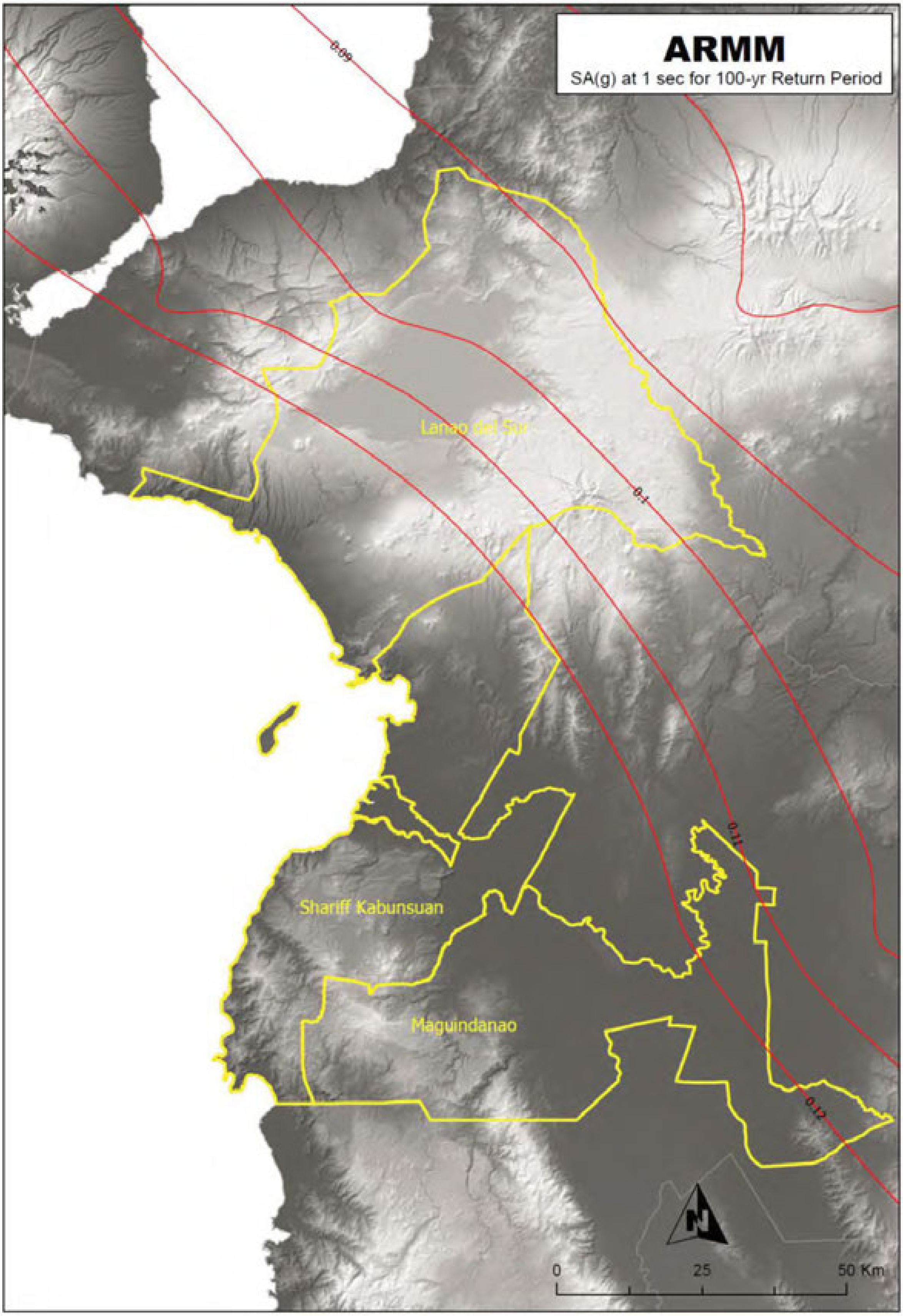


Figure 3A-53 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region ARMM)

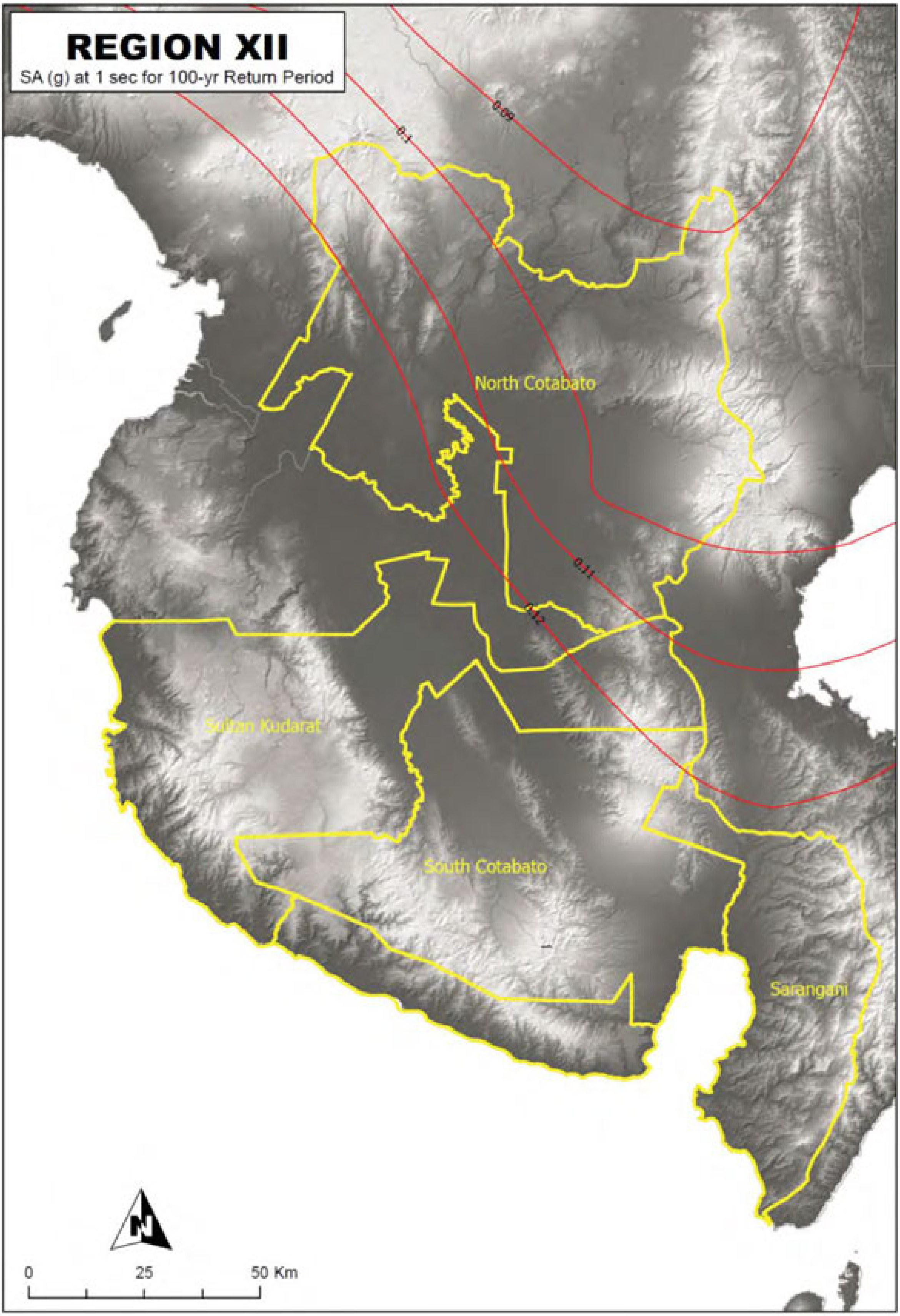


Figure 3A-54 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) for Level 1 Earthquake Ground Motion (Region XII)