PART 2

BRIDGE SEISMIC DESIGN SPECIFICATIONS (PACKAGE A)

CHAPTER 5 CHRONOLOGY OF BRIDGE SEISMIC DESIGN SPECIFICATIONS

5.1 Introduction

Although the DPWH has its own "*Design Guidelines, Criteria and Standards for Public Works and Highways*" which was first published in 1982, the seismic provisions of this guidelines has been outdated by recent earthquake events in the country and elsewhere. Owing to this deficiency in the DPWH Guidelines, the seismic design of bridges in the Philippines relies heavily on the "*American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges*" (17th Ed., 2002 using the load factor and allowable stress design) with the seismic design provisions practically guiding the design of new bridges in the Philippines. Although AASHTO's bridge design specifications have evolved to the load and resistance factor design (LRFD) and the displacement-based design procedures using probability theory and limit states, the DPWH still applies the earlier version of AASHTO in seismic design. On the other hand, seismic design of bridges in Japan focused on the performance-based approach which evolved based on the occurrences of recent large earthquakes in Japan.

This Chapter will describe the chronology of the development of the seismic design specifications for the Philippines, Japan and the USA (Figure 5.2.1-1) based on large earthquake events that led to the current state of the seismic design codes.

5.2 AASHTO Bridge Seismic Design Evolution (USA)

The AASHTO highway bridge design specifications have evolved several times during the last 80 years. The codes have developed from the allowable stress to load factor and to load and resistance factor design. The seismic design provisions likewise progressed from the equivalent static lateral force to the response spectrum method using force-based approach and recently the displacement-based approach. Since the Philippine bridge design practice basically adopts the AASHTO Standard Specifications, the following will summarize the development of the AASHTO seismic design specifications:

5.2.1 Early Design Code Stages

The first American Association of State Highway Officials (AASHO), later AASHTO, specifications for highway bridges was published in 1931, but it did not address the issue of seismic design until 1940. However, although the 1941 AASHO code required that bridges be designed for earthquake load, there is no explicit provision on how to determine the seismic load. It was in 1943 that the California Department of Transportation (Caltrans) developed various levels of equivalent static load forces for seismic design of bridges combining such for the design of members using the working stress design (WSD). However, it was not until 1961 that the edition of AASHO specifies earthquake loading for use in the WSD approach following the Caltrans' criteria.

Major EQ	Japan Seismic Design	USA Seismic Design	Philippine Seismic Design
1920			
• 1923 Kanto EQ, Japan	• 1925 1st Bridge Design Code with Seismic Provision • 1929 Draft Detailed Reg. for Road Structures		
1930			
	• 1939 Draft Specs for Highway Bridge (kh = 0.20)	• 1931 AASHO Standard Specifications for Highway Bridges(1st Ed.), • 1935 AASHTO (2nd Ed.)	• No specific seismic provision • Basically follows AASHO/AASHTO provisions
1940			
• 1948 Fukui EQ, Japan		• 1941 AASHO (3rd Ed.) • 1944 AASHO (4th Ed.) • 1949 AASHO (5th Ed.)	
1950			
	• 1956 Specification for Steel Highway Bridge (kh = 0.10-0.35, corrected by ground type)	• 1953 AASHO (6th Ed.) • 1957 AASHO (7th Ed.)	
1960			
• 1964 Niigata EQ, Japan	• 1964 Specifications for Steel Highway Bridge • 1964 Design Specifications (kh=0.2, kv=0.10)	• 1961 AASHO (8th Ed.) – EQ load applied to WSD • 1965 AASHO (9th Ed.) • 1969 AASHO (10th Ed.) - Equivalent lateral static force coefficient method (C =0.02, 0.04, 0.06) • Prior to 1971, seismic design adopted lateral forces used in buildings	
1970			
• 1971 USA San Fernando EO • 1978 Miyagi EQ, Japan	• 1971 Road Bridge Seismic Design Specs. (kh = 0.1- 0.30, with corrections) - Unseating & liquefaction provision	• 1973 AASHTO (11th Ed.) - Caltrans introduced new seismic criteria • 1977 AASHTO (12th Ed.) - Adopted Caltrans seismic design	
1980			
• 1982 Central Japan Sea EQ • 1989 USA Loma Prieta	• 1980 Specs for Highway Bridges, V - Seismic Design (modified coefficient & deformation check)	• 1983 AASHTO (13th Ed.) - ATC6 Seismic Design Guide Specs. • 1989 AASHTO (14th Ed.)	• 1982 DPWH Design Guidelines Criteria - Uses J.P. Hollings Report > kh=0.10(DL + 0.5LL) • 1987 NSCP Vol. II Bridges (1st Ed.)
1990			
• 1990 North Luzon EQ. Philippines • 1993 Kushiro EQ, Japan • 1994 USA Northridge EQ • 1995 Kobe EQ, Japan	• 1990 Specs for Highway Bridges, V - Seismic Design (Ductility, Ground Motion, kh=0.70-1.0, Residual Strength Check) • 1995 Specs. Restoration of Highway Bridges Damaged by Kobe EQ. (kh=1.5-2.0) • 1996 Specs for Highway Bridges, V - Seismic Design (Near field/inland ground motion for dynamic analysis)	• 1992 AASHTO (15th Ed.) - Division I-A Seismic Design • 1994 AASHTO LRFD Bridge Design Specifications (1st Ed.) • 1996 AASHTO (16th Ed.) • 1998 AASHTO LRFD (2nd Ed.)	• 1992 DPWH D.O.75 - To follow AASHTO latest edition • DPWH refers to AASHTO 16th Ed. • 1997 NSCP (2nd Ed.) - Based on AASHTO 16th Ed. seismic provision with 2-zone Philippine seismic map (PGA 0.2 & 0.4)
2000			
• 2004 Niigata Chuetsu EQ, Japan	• 2002 Specs for Highway Bridges, V - Seismic Design (Seismic performance definition, dynamic analysis)	• 2002 AASHTO (17th Ed.) • 2004 AASHTO LRFD (3rd Ed.) • 2007 AASHTO LRFD (4th Ed.) • 2009 Guide Specs. for LRFD Bridge Seismic Design (1st Ed)	• 2004 DPWH Design Guidelines Criteria (Draft) - Refer to AASHTO 1996 (16th Ed.) Div. I-A • 2005 NSCP Vol II Bridges - Reprint Edition
2010			
• 2011 Tohoku District Pacific Coast EQ, Japan	• 2012 Specs for Highway Bridges, V - Seismic Design (Effects of Tohoku Pacific Coast earthquake considered)	• 2010 AASHTO LRFD (5th Ed.) • 2012 AASHTO LRFD (6th Ed.) • 2011 AASHTO LRFD Bridge Seismic Design (2nd Ed.)	• NSCP 2011 LRFD Bridge Code - Draft (on review) • Revision of the DPWH Design Guidelines, Criteria and Standards (To be implemented at the end of 2012)

Figure 5.2.1-1 Evolution of Seismic Bridge Design Specifications

5.2.2 AASHO Elastic Design Approach

- In 1969, AASHO specifies an equivalent static lateral force coefficient for the design of bridges under earthquake loading (based on the Caltrans provisions). The coefficients applied to the dead load used to determine the lateral force depends on the type of foundation (C = 0.02 for spread footings with 400 kPa or more capacity, C = 0.04 for spread footings with less than 400 kPa capacity and C = 0.06 for pile foundation). Using the WSD, a 33% increase in allowable stress is allowed for member design during earthquake loading.

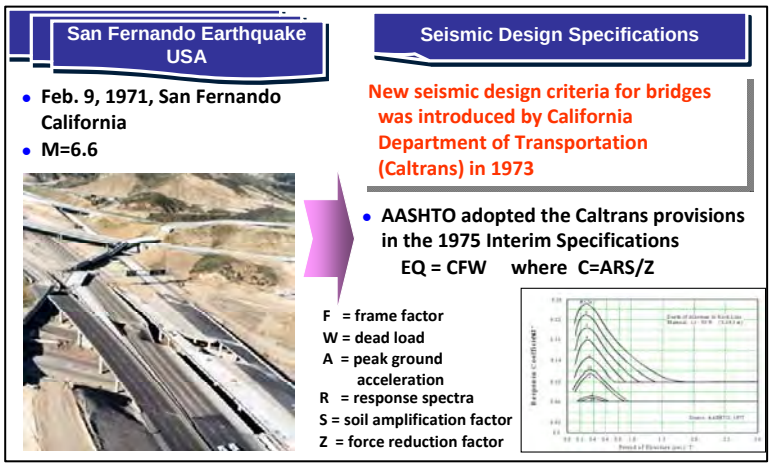


Figure 5.2.2-1 1971 San Fernando Earthquake Leading to Caltrans Seismic Provision

- However, during the 1971 San Fernando earthquake, many highway bridges were either severely damaged or collapsed despite the Caltrans seismic design provisions. The equivalent lateral force coefficients were found to be too low and columns lack ductility resulting to brittle failure. Following the lessons learned from the San Fernando earthquake, Caltrans developed the force-based seismic design procedure to include the dynamic response characteristics of the bridge and the effects of soil conditions on the seismic load.

5.2.3 AASHTO Force-Based Design Approach (WSD and LFD)

- In 1975, AASHTO adopted Caltrans seismic design approach and issued an interim specification increasing the amount of column transverse reinforcement and the girder seat widths to minimize the risk of superstructure unseating in the event of a large earthquake.
- In this approach, the equivalent static force method was also used to calculate the design earthquake loading using the response coefficient C which is a function of the expected peak ground acceleration, normalized acceleration response value for rock, soil amplification factor and force reduction factor to account for column ductility. Factors for the structural system (single column or rigid frames) were also applied.
- Options to use WSD of the load factor strength design (LFD) were provided. However, the values for the force reduction factors were not given making it difficult to determine the column ductility demand.

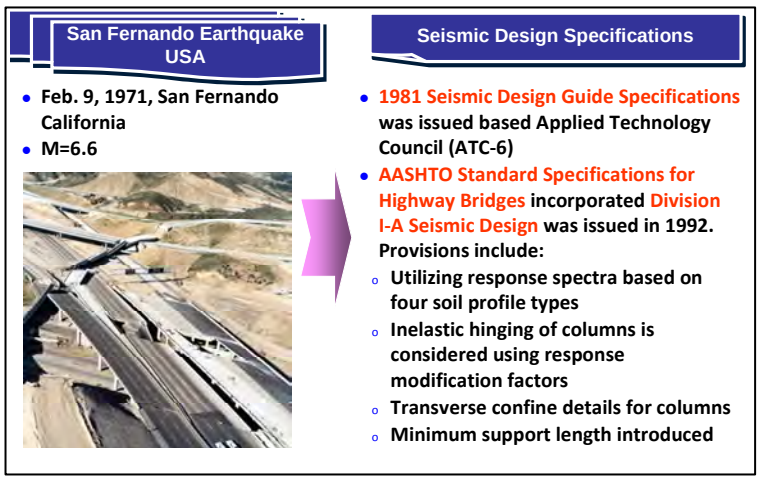


Figure 5.2.3-1 1971 San Fernando Earthquake Leading to Revision of Design Specifications

seismic design forces (Division I-A). The deviation of this edition from the previous editions include: (1) analysis of structures by response spectrum method, (2) design acceleration spectrum based on soil types, (3) elastic member forces derived from the combination of two orthogonal horizontal seismic components, (4) use of response modification factor (R) to represent column ductility demand, and (5) ductile detailing of columns with minimum transverse reinforcement. The 500-year return period earthquake was used in determining the peak ground acceleration.

5.2.4 AASHTO Force-Based Design Approach (LRFD)

- In 1994, the first edition of the *"AASHTO LRFD Bridge Design Specifications"* was published placing earthquake loading under Extreme Event I limit state. Similar to the 1992 edition, the LRFD edition accounts for column ductility using the response modification R factors. The bridge importance became three levels – "critical", "essential" and "others" where critical bridges must remain open to all traffic after the design earthquake. Moreover, bridges are assigned to seismic zones to reflect the requirements for methods of analysis and bridge details. Similarly, the elastic seismic forces are calculated by the response spectrum analysis.

- In 2008, the *"AASHTO LRFD Interim Bridge Specifications"* was published to incorporate more realistic site effects based on the 1989 Loma Prieta earthquake in California. Moreover, the elastic force demand is calculated using the 1,000-year maps as opposed to the earlier 500-year return earthquake. The design response spectrum in this interim specification is calculated using the maps of peak ground acceleration, short period (0.2s) design earthquake response spectral acceleration coefficients and the 1-sec period response spectral acceleration coefficient. The R-factor concept based on the equal-displacement approximation is used to determine the structure deformations.

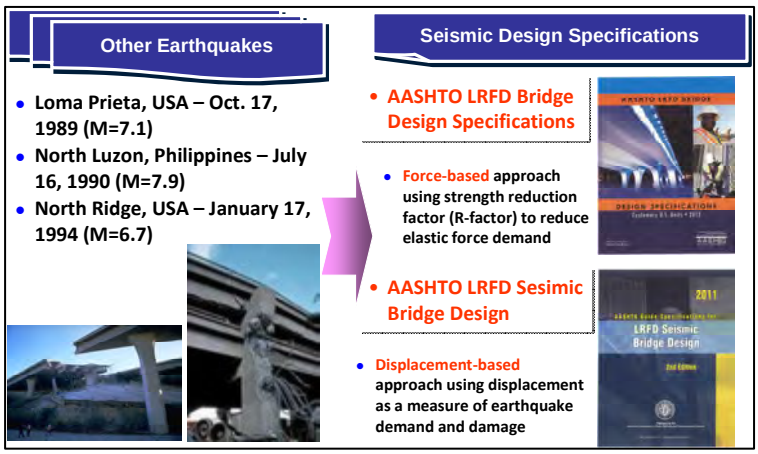


Figure 5.2.4-1 Force-based and Displacement-based AASHTO Specifications

5.2.5 AASHTO LRFD Seismic Bridge Design

- In 2009 (after the devastating earthquakes of 1989 Loma Prieta, 1990 North Luzon, 1994 Northridge, 1995 Kobe, etc.), AASHTO published the "*Guide Specifications for LRFD Seismic Bridge Design*" shifting the design focus from the force-based R-factor design approach to the displacement-based design approach to incorporate the displacement design principles for the design of ductile members.
- Improvements in the specifications include discontinuing use of the R-factors for ductile column, inelastic displacement demand in short-period structures is increased by a modification factor, use of four seismic design categories for analysis, design details and liquefaction consideration, capacity protection principles for column and cap-beam column connections and use of non-linear pushover analysis to evaluate displacement capacities.

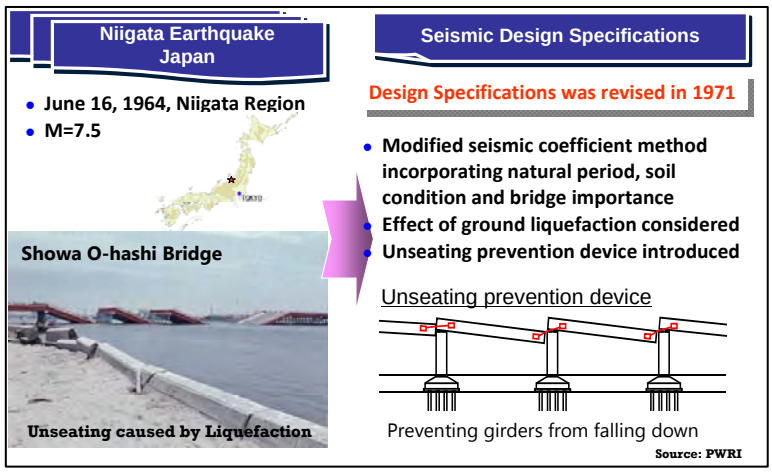


Figure 5.2.5-1 Design considerations for soil liquefaction and unseating device

5.3 Japan Bridge Seismic Design Evolution

Although the Philippine bridge design employs mainly the AASHTO Standard Specifications, reference is also made to the design procedures of the Japan Road Association (JRA) especially the analysis and design of foundations. In this regard, the evolution of the seismic design of bridges in Japan is presented as follows:

5.3.1 Early Stages of Bridge Design

- The first seismic design specifications for bridges in Japan was established in 1925 after the devastating effects of the 1923 Kanto earthquake in Tokyo. Prior to the 1923 Kanto earthquake, seismic effect was not considered or poorly considered with bridge collapse resulting due to foundation failures (instability of clayey soil and liquefaction of sandy soil).
- As a consequence of the extensive damages to bridge structures, the elastic seismic design used the equivalent static seismic coefficients of 0.2-0.3 based on the allowable stress design approach. This resulted in the construction of massive and rigid piers.

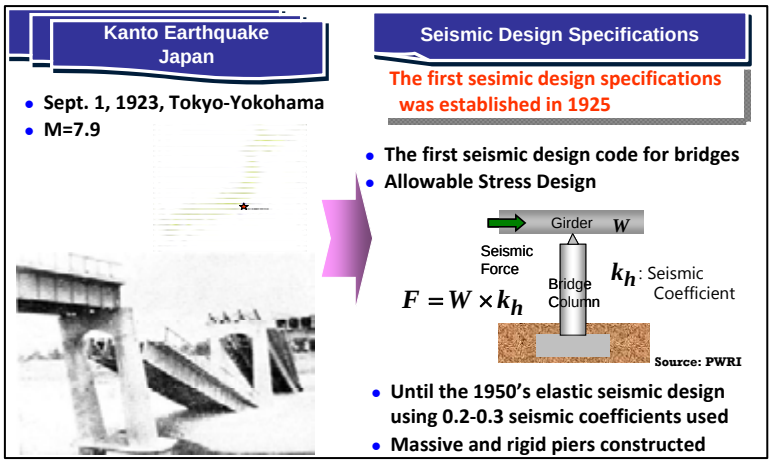


Figure 5.3.1-1 Early Stage of Japan Bridge Design

5.3.2 Consideration for Soil Liquefaction and Unseating Device

- The effects of the 1964 Niigata earthquake brought the importance of considering soil liquefaction and unseating prevention devices in seismic design of bridges. A seismic design specification was issued using the equivalent static horizontal coefficient of k_h=0.2 and vertical coefficient of k_v=0.10.
- In 1971 the "*Seismic Design Specifications for Highway Bridges*" was published incorporating (1) the modified seismic coefficient design method which considers the natural period, soil condition and bridge importance, (2) evaluation for vulnerability to liquefaction, and (3) use of unseating prevention devices.

5.3.3 Column Ductility, Bearing Strength and Ground Motion

- Insufficient consideration in column ductility and insufficient bearing strength were observed after the 1978 Miyagi earthquake, 1982 Urakawa earthquake and the 1993 Hokkaido-Toho earthquake. Premature shear failures were observed in columns, especially in areas with insufficient development length of reinforcements.
- In 1980, the "*Specifications for Highway Bridges, Part V – Seismic Design*" was published incorporating the modified design coefficient and the check for structure deformation. It was then revised in 1990 to include check for column ductility and residual deformations, lateral forces for multi-span bridges and the standard ground motions for dynamic analysis.
- After the devastating effects of the Kobe earthquake in 1995 that brought about collapse of major bridges due to insufficient strength and ductility of columns, bearings and unseating prevention devices, the "*1995 Specifications for Restoration of Highway Bridges Damaged by Kobe Earthquake*" was issued. Moreover, the "*Specifications for Highway Bridges, Part V – Seismic Design*" was revised in 1996 to introduce a two-level performance design concept and include the near-field ground motion and column ductility design improvement.

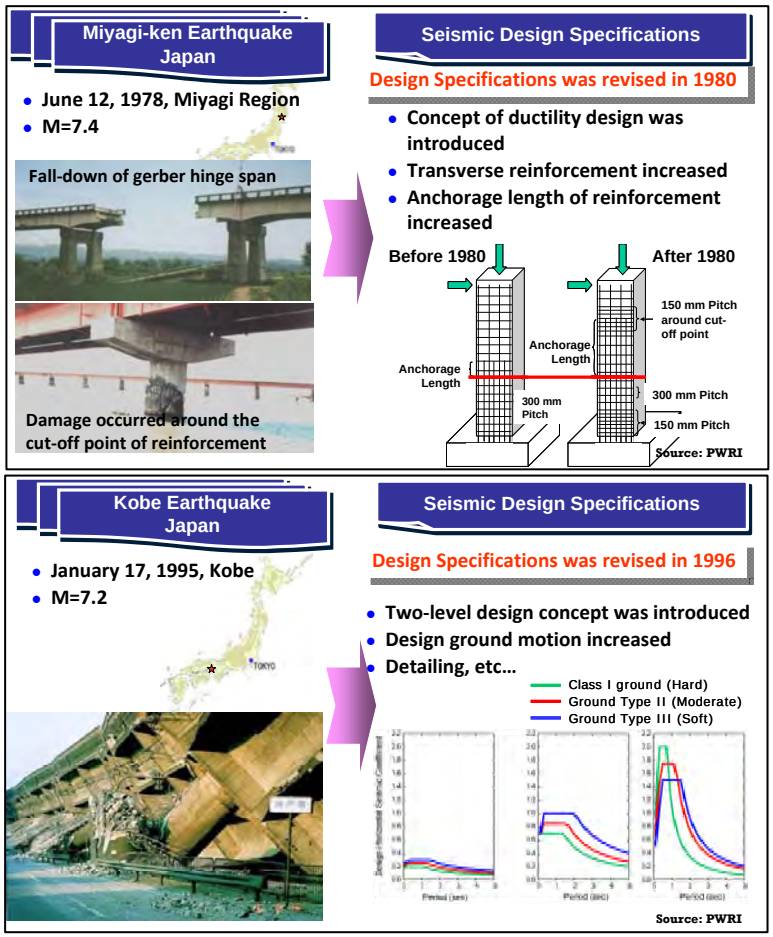


Figure 5.3.3-1 Column Ductility Design and Near-Field Ground Motion

- In 2002, further revisions to the specifications were undertaken to include the definitions for seismic performance and guidance for dynamic analysis of bridges.
- Considering the effects of the 2011 Tohoku Pacific Coast earthquake, the specification was further revised in 2012.

5.4 Philippine Seismic Bridge Design Evolution

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.

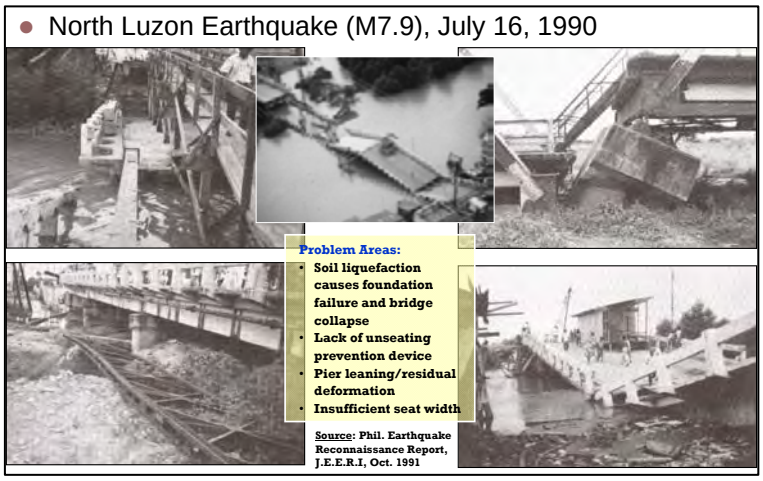


Figure 5.4-1 North Luzon Earthquake, 1990 (Philippines)

- 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.

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

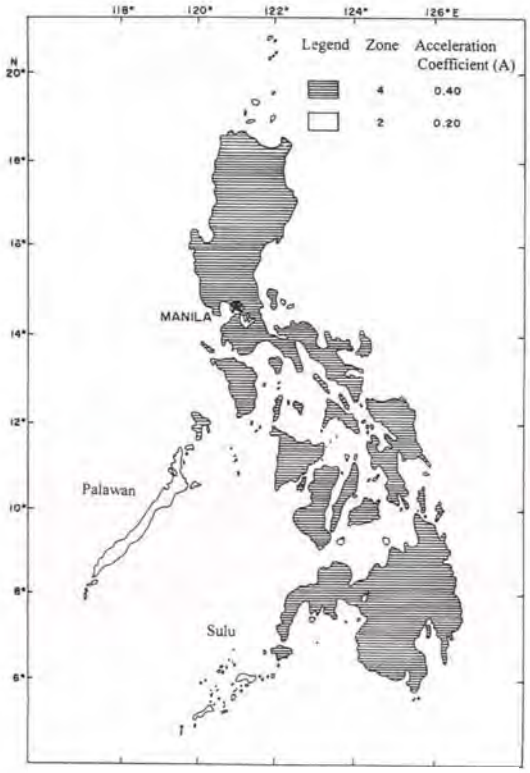


Figure 5.4-2 Philippine Seismic Zone Map

- In 1990, the North Luzon earthquake caused major damages to public infrastructure in the Philippines including collapsed of highway bridges due to soil liquefaction, lack of unseating prevention device and insufficient seat width. Most bridges damaged by the earthquakes are those designed with minimal or no considerations for earthquake forces.
- Due the urgency of the need to establish proper seismic design considerations for bridges, the DPWH issued D.O.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.
- 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, 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 remains a draft.
- At present, the DPWH still refer to the ASEP seismic zone map for the ground 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.
- 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 will undertake the project "*Enhancement of Management and Technical Processes for Engineering Design in the DPWH"* under the National Road Improvement and Management Program 2 (NRIMP-2). One component of this project is to develop the new Design Guidelines, Criteria and Standards.

CHAPTER 6 COMPARISON ON BRIDGE SEISMIC DESIGN SPECIFICATIONS BETWEEN DPWH/NSCP, AASHTO AND JRA

6.1 Purpose of Comparison

Since the seismic loading provisions of the DPWH Design Guidelines (1982) have been outdated by recent earthquake events, the current seismic design of bridges practiced by DPWH under D.O.75 requires, as a minimum, that bridge design shall conform to the AASHTO Guide Specifications for Seismic Design (1989 or latest edition). This seismic design provision (with reference to AASHTO 1992 (15th Ed.) is applied by ASEP in the NSCP Vol. 2 – Bridges, using the allowable stress design with the load factor design. The latest edition of NSCP is the 2005 reprint of the 2nd edition (1997).

However, since the issuance of D.O.75, the AASHTO seismic design have evolved from the working stress and load factor design to load and resistance factor design using the force-based procedures (AASHTO 6th Edition, 2012) and the displacement-based procedures (AASHTO 2nd Edition, 2011 Seismic Bridge Design) to calculate the elastic demand forces and the member ductility demand. Several large earthquakes occurring in the U.S.A. and elsewhere prompted the AASHTO to modify the seismic design provisions as explained in Chapter 5.

Likewise, the Japan Road Association (JRA) Seismic Design Specifications for Highway Bridges evolved as a result of the data accumulated and lessons learned from recent major earthquakes, with the latest revision being the acceleration response spectra for Type I design earthquake due to the 2011 Tohoku District Pacific Coast earthquake.

In order to realize the differences in seismic design requirements and procedures between the DPWH/NSCP, the AASHTO and the JRA Specifications, this Chapter compares the following recent specifications for seismic design of bridges:

- NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition, ASEP, (Reference to AASHTO 1992 15th Ed.). This code will be used in the comparison since the DPWH Guidelines is superseded by the DPWH D.O.75 which also refer to the AASHTO design procedures.
- AASHTO LRFD Bridge Design Specifications (2012, 6th Ed.) – Force-based R-factor Method
- AASHTO Guide Specifications for LRFD Seismic Bridge Design (2011, 2nd Ed.) – Displacement-based Method
- JRA Part V – Seismic Design (2012 Ed.)

6.2 Items for Comparison

The comparison of the codes and specifications mentioned in Section 7.1 is based on the following items:

- Principles of Seismic Design (Section 6.3.1) – the philosophy and principles of seismic design applied in the specifications
- Seismic Performance Requirements (Section 6.3.2) – the expected performance of bridges from small to moderate to large earthquake
- Design Procedures and Methods (Section 6.3.3) – design procedures and methods applicable for seismic performance verification
- Acceleration Response Spectra (Section 6.3.4) – ground acceleration and acceleration response spectra used to determine elastic force demands and ductility demands of members
- Unseating/Fall-Down Device (Section 6.3.5) – provisions for unseating or fall-down of superstructures
- Foundation Design (Section 6.3.6) – provisions for design of foundations under seismic loading

6.3 Difference in Major Items between NSCP, AASHTO and JRA

6.3.1 Principles of Seismic Design

The following compares the provisions of the codes and specifications on the design philosophy and concept:

DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition
The principles used in the development of the provisions are: 1) Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. 2) Realistic seismic ground motion intensities and forces are used in the design procedures. 3) Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Development of the NSCP Standards is based on the following concepts: • Hazard to life be minimized, • Bridges may suffer damage but have low probability of collapse due to earthquake motions, • Function of essential bridges be maintained, • Design ground motions have low probability of being exceeded during normal life of bridge, • Provision be applicable to all parts of the Philippines, and • Ingenuity of design not be restricted. Department Order No. 75 (D.O.75), July 17, 1992 "*DPWH Advisory for Seismic Design of Bridges*" requires the following design concept to be adopted: • Continuous bridges with monolithic multi-column bents have a high degree of redundancy and are 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.

- Where multi-span simple span bridges are justified, decks should be continuous.
- Restrainers (horizontal linkage device between adjacent spans) are required at all joints in accordance with AASHTO provisions and generous seat widths at piers and abutments should be provided to prevent loss-of-span type of failures.

- Transverse reinforcement in the zones of yielding is 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 presenting buckling of the main reinforcement.
- 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.
- The stiffness of the bridge as a whole should be considered in the analysis. In regular structures, as defined previously, it is particularly important to include the soil-structure interaction.

AASHTO LRFD Bridge Design Specifications, 2012, 6th Ed.

The following principles are applied in the development of the specifications:

- Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics.
- Each component and connection shall satisfy the requirements of force effect and resistance for each limit state.
- The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake.
- The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure. Energy dissipating devices may be substituted for conventional ductile earthquake resisting systems. In order to achieve adequate inelastic behavior, the system should have sufficient number of ductile members and either:
  - Joints and connections that are also ductile and can provide energy dissipation without loss of capacity; or
  - Joints and connections that have sufficient excess strength so as to assure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.
- Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them.
- The possibility of partial live load with earthquakes, especially in urban areas, should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that a factor of 0.5 is reasonable for a wide range of values of average daily truck traffic (ADTT).

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

- The Guide Specifications are approved as an alternate to the seismic provisions in the AASHTO LRFD Design Specifications. These Guide Specifications differ from the current procedures in the LRFD Specifications in the use of "***displacement-based***" design procedures, instead of the traditional, "***force-based R-factor***" method.
- The key features of these Guide Specifications follow:
  - Adopt the seven percent in 75 year design event for development of a design spectrum.
  - Adopt the NEHRP Site Classification system and include site factors in determining response spectrum ordinates.
  - Ensure sufficient conservatism (1.5 safety factor) for minimum support length requirement. This conservatism is needed to accommodate the full capacity of the plastic hinging mechanism of the bridge system.
  - Establish four Seismic Design Categories (SDCs) A, B, C and D.
  - Allow three types of bridge structural system:
    - Type 1 – Design a ductile substructure with an essentially elastic superstructure.

- Type 2 – Design an essentially elastic substructure with a ductile superstructure.
- Type 3 – Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.

- Critical/essential bridges are not specifically addressed in these Guidelines. Classification of critical/essential bridges includes:
  - Bridges that are required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles and for security, defense, economic, or secondary life safety purposes immediately after the design earthquake.
  - Bridges that should, as a minimum, be open to emergency vehicles and for security, defense or economic purposes after the design earthquake and open to all traffic within days after the event.
  - Bridges that are formally designated as critical for a defined local emergency plan.
- Seismic effects of box culverts and buried structures need not be considered except where failure of the box culvert or buried structures will affect the bridge.
- Adjacent bents within a frame or adjacent columns within a bent shall have an effective stiffness ratio equal to or greater than 0.75 while any two bents within a frame or any two columns within a bent shall have an effective stiffness ratio equal to or greater than 0.5.
- The ratio of the fundamental periods of vibration (less flexible to more flexible) for adjacent frames in the longitudinal and transverse direction shall be equal to or greater than 0.75.

JRA Part V – Seismic Design, 2012 Ed.

- Two levels of design earthquake ground motions shall be considered in the seismic design of a bridge. The first level corresponds to an earthquake with high probability of occurrence during the bridge service life (called “Level 1 Earthquake Ground Motion”) and the second level corresponds to an earthquake with less probability of occurrence during the bridge service life but strong enough to cause critical damage (called “Level 2 Earthquake Ground Motion”). For the Level 2 Earthquake Ground Motion, two types of earthquake ground motion shall be taken in to account, namely, Type I plate boundary type earthquake with large magnitude and Type II inland direct strike type earthquake.
- Seismic performance of bridges shall have three levels during earthquakes, – keeping its sound function sustaining limited damage and sustaining no critical damage.
- Bridges shall be designed so that unseating of superstructures can be prevented, even though structural failures may occur due to structural behavior or ground failure unexpected in the seismic design.
- It shall be ensured that the seismic performance according to the levels of design earthquake motion and the importance of a bridge.
- It is desirable to adopt a multi-span continuous structure, the type of which bearing supports is to be a horizontal force distributed structure.
- It is generally better for a bridge with tall piers built in a mountainous region to resist seismic horizontal forces by abutments rather than piers if the ground conditions at the abutments are sufficiently sound.(The seismic performance of the whole bridge should be considered, and proper bearing supports in view of bridge structural conditions and ground bearing properties should be selected.)
- On reclaimed land or alluvial ground where ground deformation such as sliding of a soft cohesive clayey layer, liquefaction of sandy layer and liquefaction-induced ground flow may happen, a foundation with high horizontal stiffness should be designed, and a structural system such as multi-fixed-point type and rigid frame type, which has many contact points between the superstructure and substructure, should be selected.
- A seismically-isolated bridge should be adopted for a multi-span short-period continuous bridge on stiff ground conditions.
- For a strong earthquake motion, a proper structural system shall be designed by clarifying structural members with nonlinear behavior and those basically remaining in elastic states.
- A structure greatly affected by geometrical nonlinearity or a structure having extensive eccentricity of dead loads, which have tends to become unstable during a strong earthquake motion, shall not be adopted.
- When ground conditions or structural conditions on a pier change remarkably, whether a case of two girder ends or that of a continuous girder is more advantageous is carefully examined.

6.3.2 Seismic Performance Requirements

The level of bridge performance in the event of earthquake occurrence is important in view of the seismic behavior of bridges considering safety, serviceability and repairability for seismic design.

DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition

The following summarizes the performance level required of bridges:

Earthquake Level	*Bridge Types*	*Serviceability Performance*	*Safety Performance*
Small/Moderate	Conventional and regular bridge types	• Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.	• No significant damage to members
Large/Major (500-year return)	Critical bridges/ Essential bridges/ Conventional and regular bridges	• No explicit performance criteria but since collapse is not allowed and damages can be repaired, bridges are expected to function after the design earthquake event.	• May suffer damage but should not cause collapse of all or any of its parts. • Damage should be readily detectable and accessible for inspection and repair.

AASHTO LRFD Bridge Design Specifications, 2012, 6th Ed.

"Bridges shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a seven percent probability of exceedance in 75 years. Partial or complete replacement may be required".

Earthquake Level	*Bridge Types*	*Serviceability Performance*	*Safety Performance*
Small/Moderate	Conventional and regular bridge types	• Should resist earthquakes within the elastic range of the structural components	• No significant damage
Large/Major (1,000-year return)	Critical bridges	• Required to be open to all traffic once inspected after seismic event and usable by emergency vehicles for security, defense, economic or secondary life safety purposes immediately after the seismic event • Should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the seismic event and open to all traffic within days after that event.	• May suffer damage but with low probability of collapse.
	Essential bridges		
	Conventional/ regular bridge and less important bridges	• May suffer significant damage and disruption to service	

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

Life safety for the design event shall be taken to imply that the bridge has a low probability of collapse but may suffer significant damage and that significant disruption to service is possible. Partial or complete replacement may be required.

Earthquake Level	***Bridge Types***	***Serviceability Performance***	***Safety Performance***
Moderate	Conventional bridge types	• Should resist earthquakes within the elastic range of the structural components	• Minimal damage
Large/Major (1,000-year return)	Conventional bridge types	• Significant disruption to service shall be taken to include limited access (reduced lanes, light emergency traffic) on the bridge. • May include limited offsets and displacements.	• May suffer significant damage but with low probability of collapse. • Significant damage shall be taken to include permanent offsets and damage consisting of: - Cracking, - Reinforcement yielding, - Major spalling of concrete, - Extensive yielding and local buckling of steel columns, - Global and local buckling of steel braces, and - Cracking in the bridge deck slab at shear studs. • Partial or complete replacement of columns may be required. • For sites with liquefaction, liquefaction-induced lateral flow, or liquefaction-induced lateral spreading, inelastic deformation may be permitted in piles and shafts. Partial and complete replacement of columns, piles or shafts may be necessary.
	Critical bridges/ Essential bridges	• Required to be open to all traffic once inspected after seismic event and usable by emergency vehicles for security, defense, economic or secondary life safety purposes immediately after the seismic event • Should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the seismic event and open to all traffic within days after that event. • Significant disruption to service is possible or closure to repair the damage.	• Life safety with low probability of collapse but may suffer damage that is readily accessible for repair.

JRA Part V – Seismic Design, 2012 Ed.

- Seismic performance of bridges is taken as:

Seismic Performance	***Seismic Safety Design***	***Seismic Serviceability Design***	***Seismic Reparability Design***	
			Emergency Reparability	***Permanent Reparability***
Level 1 Keeping the sound functions of bridges	To prevent girders from unseating	To ensure the normal functions of bridges (within elastic limit states)	No repair work is needed to recover the functions	Only easy repair works are needed
Level 2 Limited damages and recovery	Same as above	Capable of recovering functions within a short period after the event	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair works
Level 3 No critical damages	Same as above	-	-	-

**: "-": Not covered*

(2) Relationship between Design Earthquake Ground Motions and Seismic Performance of Bridges

Levels of Earthquake Ground Motions		Class A Bridges*	Class B Bridges*
Level 1: Highly probable during the bridge service life		**Seismic Performance Level 1** is required	
Level 2	Type I: An Plate Boundary Type Earthquake with a Large Magnitude	**Seismic Performance Level 3** is required	**Seismic Performance Level 2** is required
	Type II: An Inland Direct Strike Type Earthquake		

**: Class A Bridges: Standard Importance; Class B Bridges: High Importance (Class A and B are classified according to such importance factors as road class, bridge functions and structural characteristics). When bridge importance is classified in view of the roles expected in the regional disaster prevention plan and road serviceability, the following should be considered.*

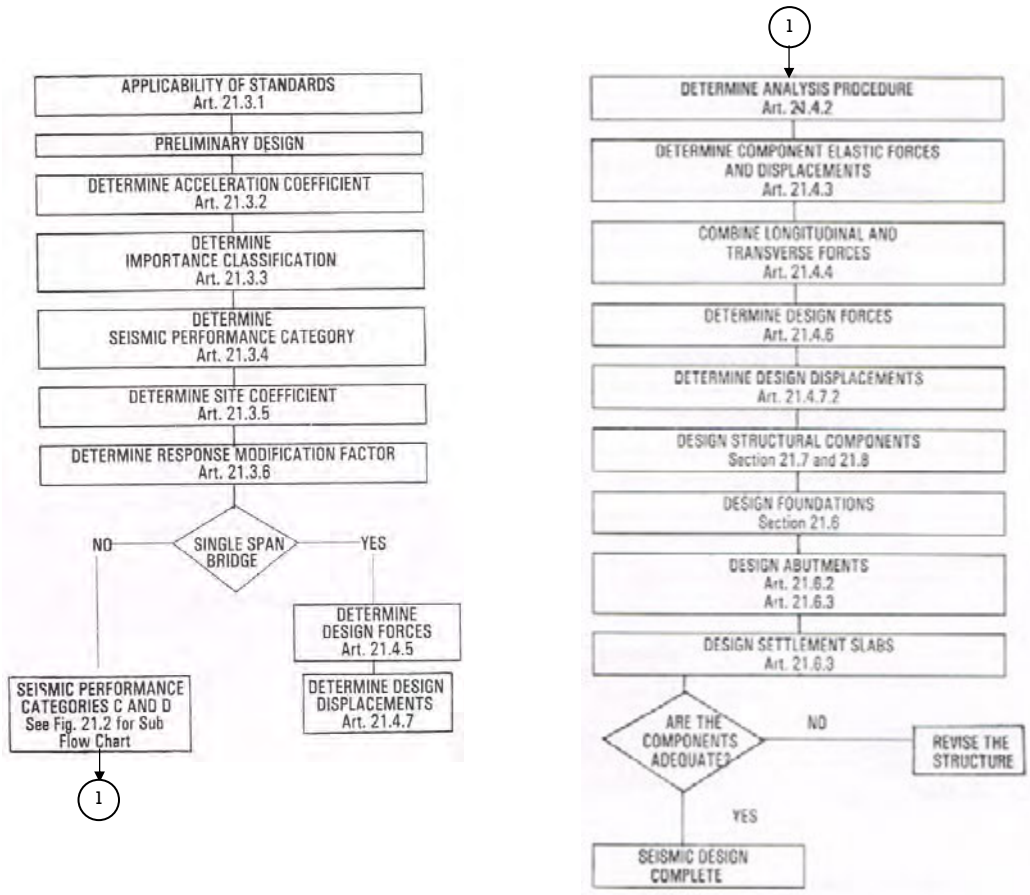
(a) To what extent a bridge is necessitated for post-event rescue and recovery activities as emergency transportation routes.
(b) To what extent damages to bridges (such as double-deck bridges and overbridges) affect other structures and facilities.
(c) Present traffic volume of the bridge and availability of substitute in case of the bridges losing pre-event functions.
(d) Difficulty (duration and cost) in recovering bridge function after the event.

6.3.3 Design Procedures and Methods

Verification of the bridge compliance with the desired performance level is done using various analysis and design methods. This section compares the procedures and methods of analysis and design of the four specifications.

DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition

(1) The procedure for seismic design is illustrated below:



(a) Design Flow Chart

(b) Sub-flow chart for Seismic Performance Categories C and D

Seismic Design Procedure Flow Chart

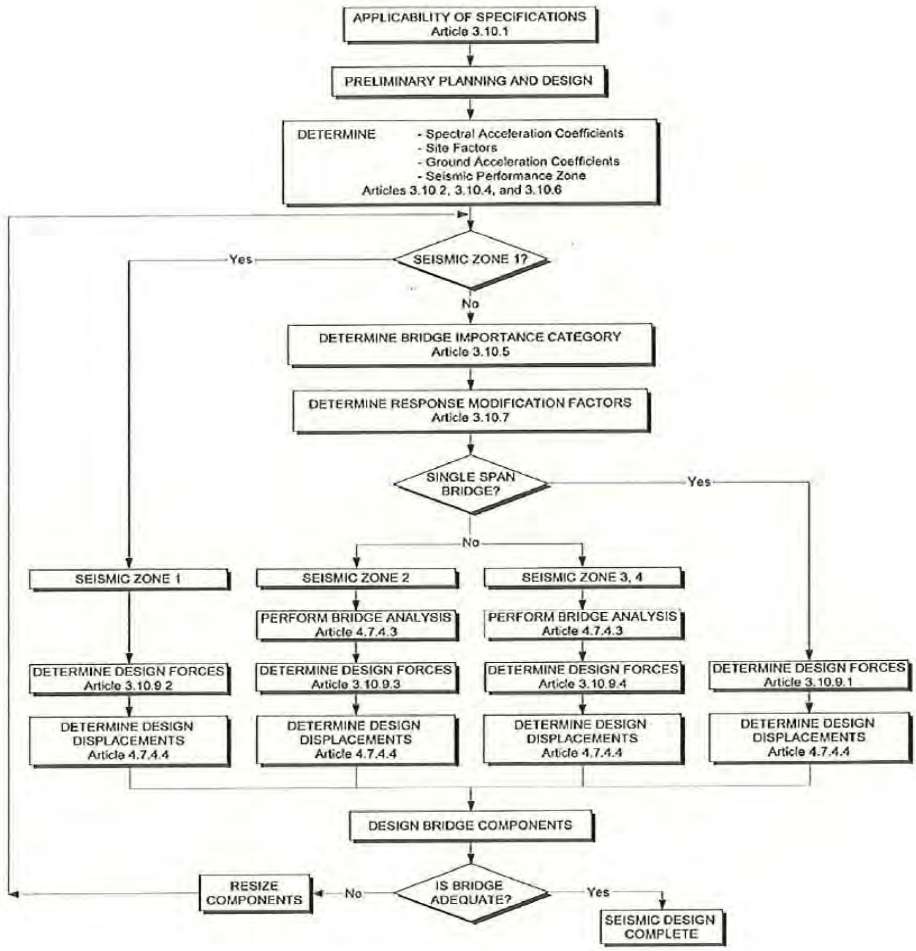
(2) The method for seismic analysis is based on:

- Two analysis methods are recommended:
  Procedure 1: Single-Mode Spectral Method
  Procedure 2: Multimode Spectral Method
- The analysis procedures above depends on the geometrical complexity of the bridge and the seismic performance category given below:

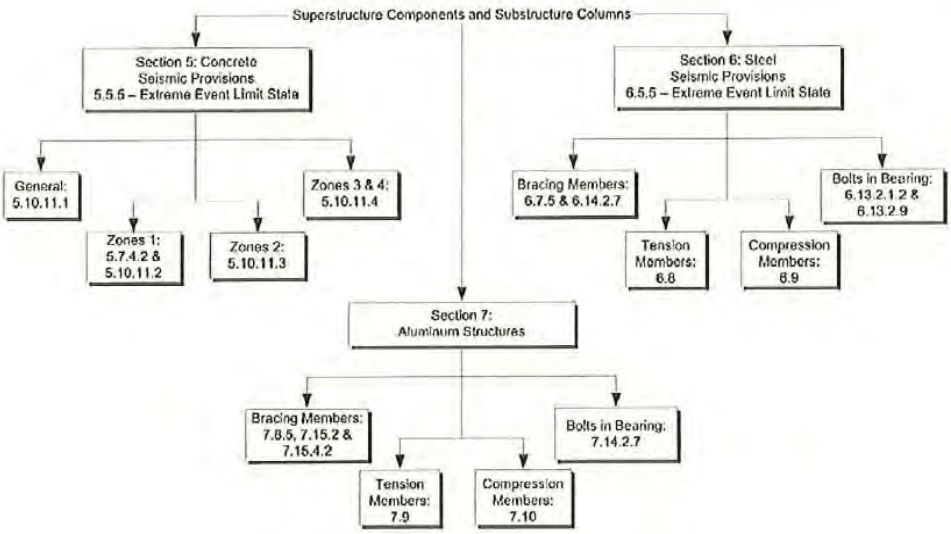
Seismic Performance Category	Regular bridges with 2 or more spans	Irregular bridges with 2 or more spans
C	1	2
D	1	2

- In both methods, all fixed column, pier or abutment supports are assumed to have the same ground motion at the same instant of time. At movable supports, displacements determined from the analysis which exceeded the minimum requirements shall be used in the design without reduction.
- Mathematical Model. The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia effects of the structure. The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included (Generally, the inertia effects of live loads are not included in the analysis; however, the design of bridges having high live to dead load ratios located in metropolitan areas where traffic congestion is likely to occur should consider the probability of large live load being on the bridge during an earthquake).
- Superstructure. The superstructure should, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to the joints at the ends of each span. The effects of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.
- Substructure. The intermediate columns or piers should also be modeled as space frame members. The model should consider the eccentricity of the columns with respect to superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear spring coefficients.
- Mode Shapes and Periods. The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by established methods for the fixed base condition using the mass and elastic stiffness of the entire seismic resisting system. The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes.
- The member forces and displacement can be estimated by combining the respective response quantities (e.g. force, displacement or relative displacement) from the individual modes by the Square Root of the Sum of the Squares (SRSS) method. For bridges with closely spaced modes (within 10%), other more appropriate methods of combining or weighting the individual contributions should be considered to obtain the total final response.

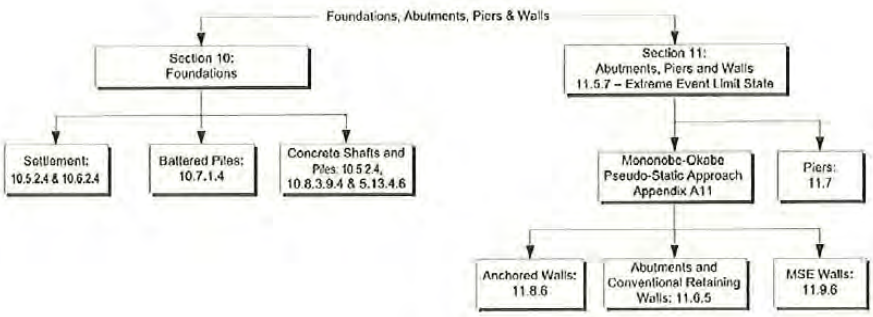
(1) The procedure for seismic design is illustrated below:



Seismic Design Procedure Flow Chart



Seismic Detailing Flow Chart



Foundation Design Flow Chart

(2) The method for seismic analysis is based on:

- It should be demonstrated that a clear, straightforward load path to the substructure exists and that all components and connections are capable of resisting the imposed load effects consistent with the chosen load paths. A viable load path shall be established to transmit lateral loads to the foundation based on the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing.
- The selection of the method of analysis depends on seismic zone, regularity, and operational classification of the bridge. Minimum analysis requirements for seismic effects are specified in the Table below:

Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single Span Bridges	Multi-span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No detailed seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

Notes * = *no detailed seismic analysis required*
UL = *uniform load elastic method*
SM = *single-mode elastic method*
MM = *multi-mode elastic method*
TH = *time history method*

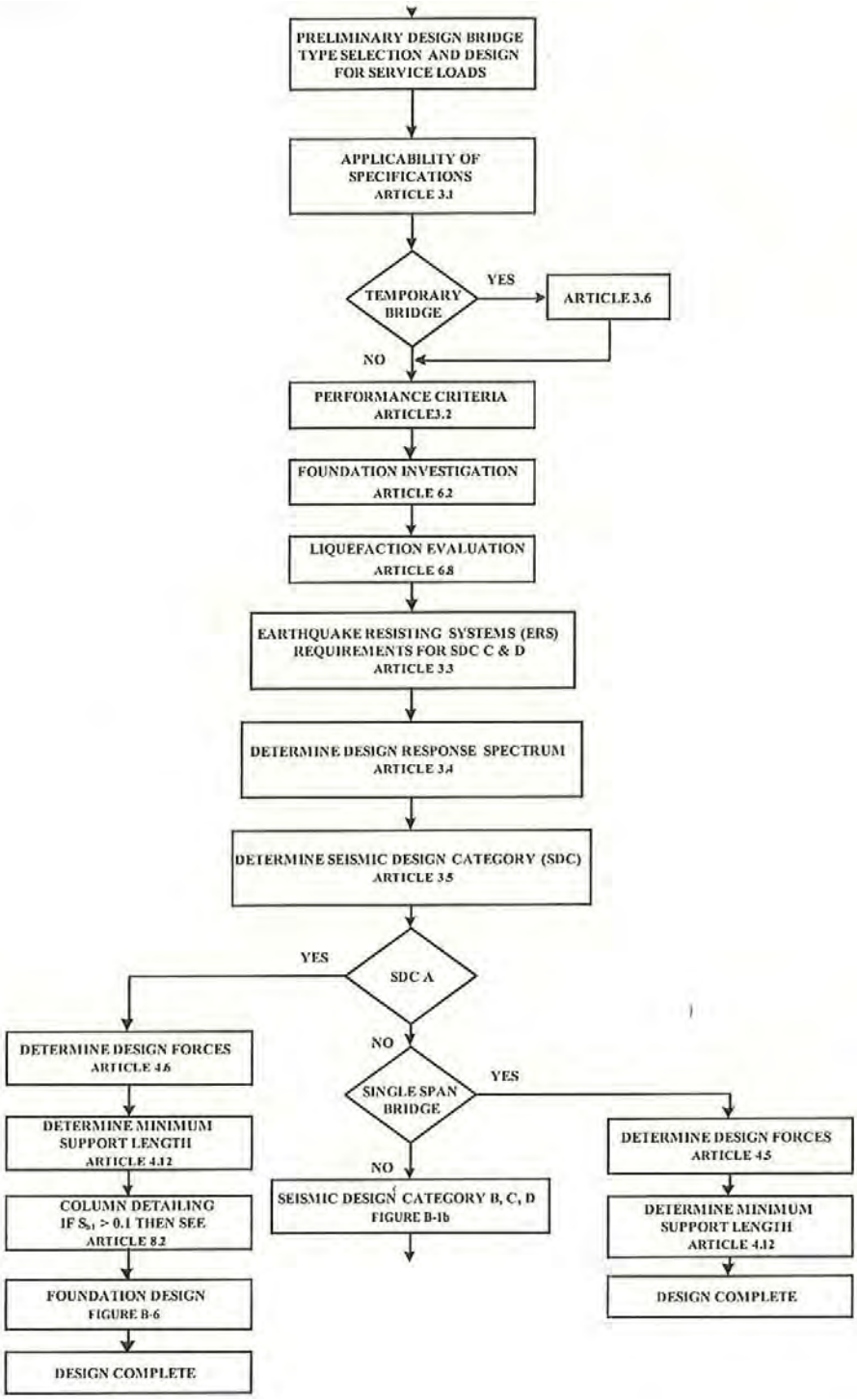
- The requirements to satisfy as regular bridges are given in the next Table, otherwise it shall be taken as "irregular" bridges.

Regular Bridge Requirements

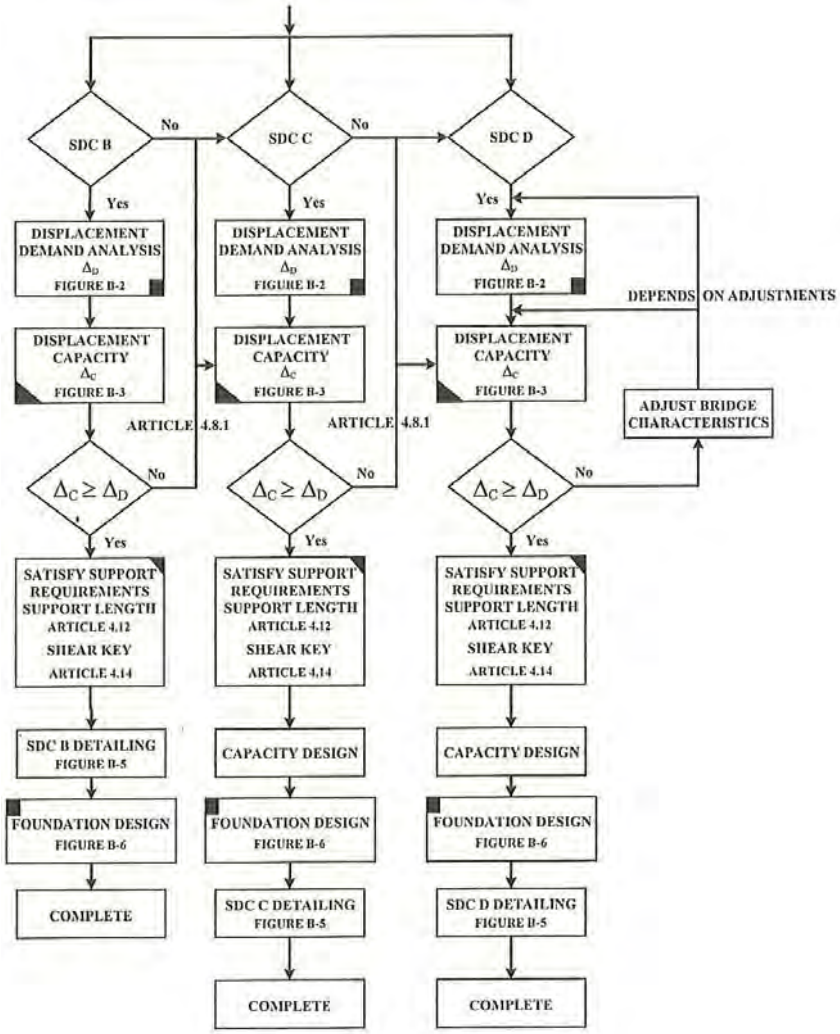
Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutment	-	4	4	3	2

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

(1) The procedure for seismic design is illustrated below:



Seismic Design Procedure Flow Chart



Seismic Design Procedure Flow Chart

(2) The method for seismic analysis is based on:

- Each bridge shall be assigned to one of four seismic design categories (SDCs), A through D, based on 1-sec period design spectral acceleration for the design earthquake (S_{D1}).
- If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed with SDC D, regardless of the magnitude S_{D1}.

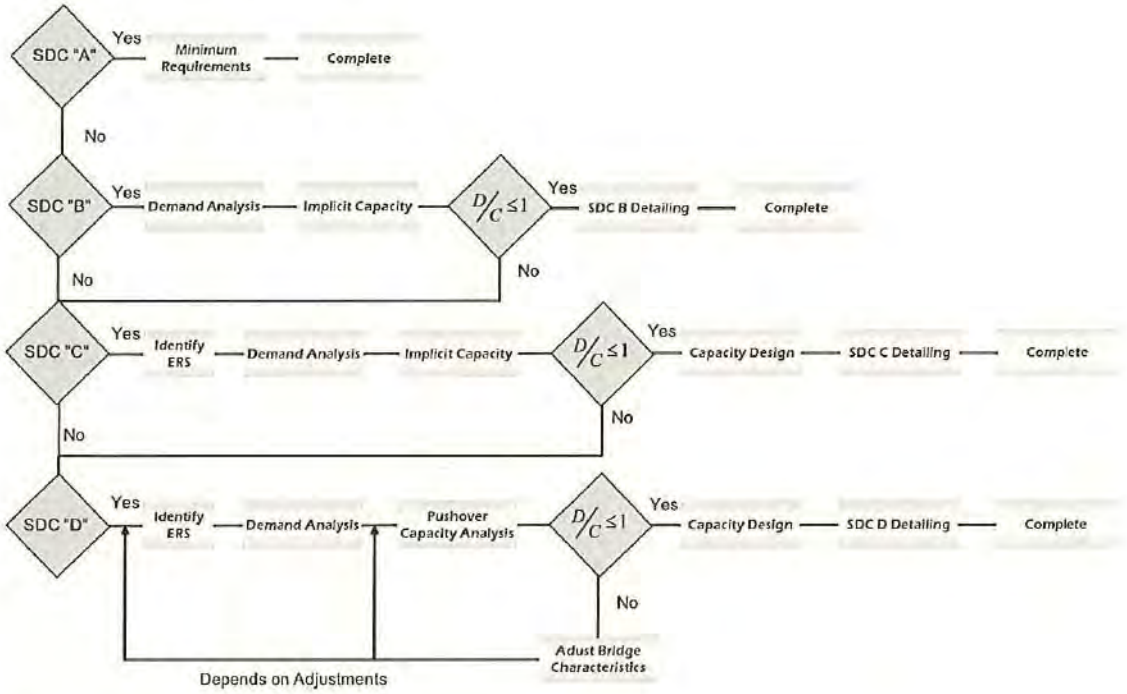
Seismic Design Categories

Value of $S_{D1} = F_v S_1$	**SDC**
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

- The requirements for each of the proposed SDCs shall be taken as shown in figure below.

SDC Requirements

Design Requirements	Seismic Design Category (SDC)			
	A	**B**	**C**	**D**
1. Identification of Earthquake Resisting System (ERS)	Not required	To be considered	Required	Required
2. Demand Analysis	Not required	Required	Required	Required
3. Capacity Check	Implicit capacity check not required	Implicit capacity check required (displacement, P-Δ, support length)	Implicit capacity check required (displacement, P-Δ, support length)	Required (displacement by pushover analysis, P-Δ, support length)
4. Capacity design	Not required	To be considered for column shear; considered to avoid weak links in the ERS	Required including column shear requirement	Required
5. Detailing Level	Minimum detailing for support length, superstructure/ substructure connection design force, column and transverse steel	SDC B level	SDC C level	SDC D level
6. Liquefaction Evaluation	Not required	To be considered for certain conditions	Required	Required



Seismic Design Category (SDC) Core Flowchart

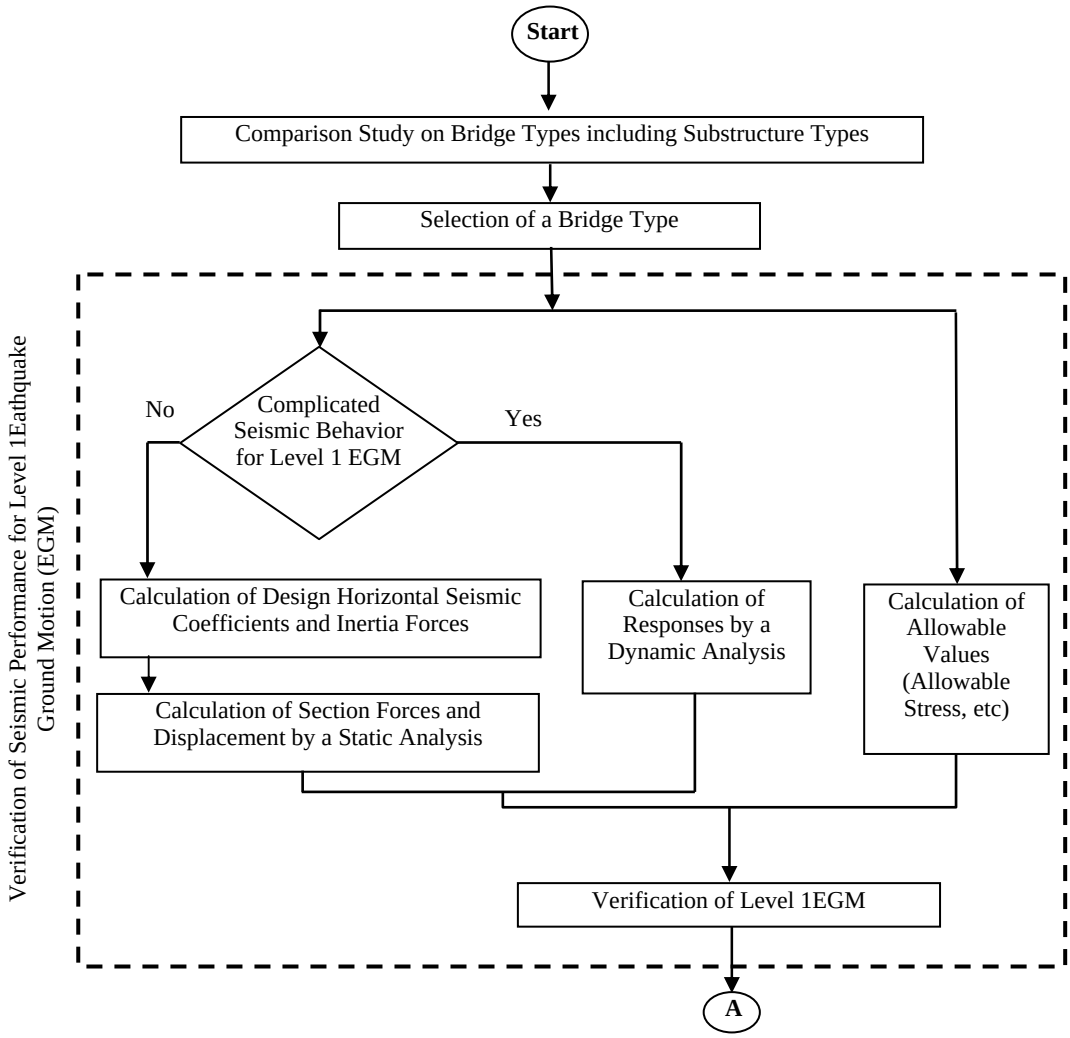
Analysis Procedures

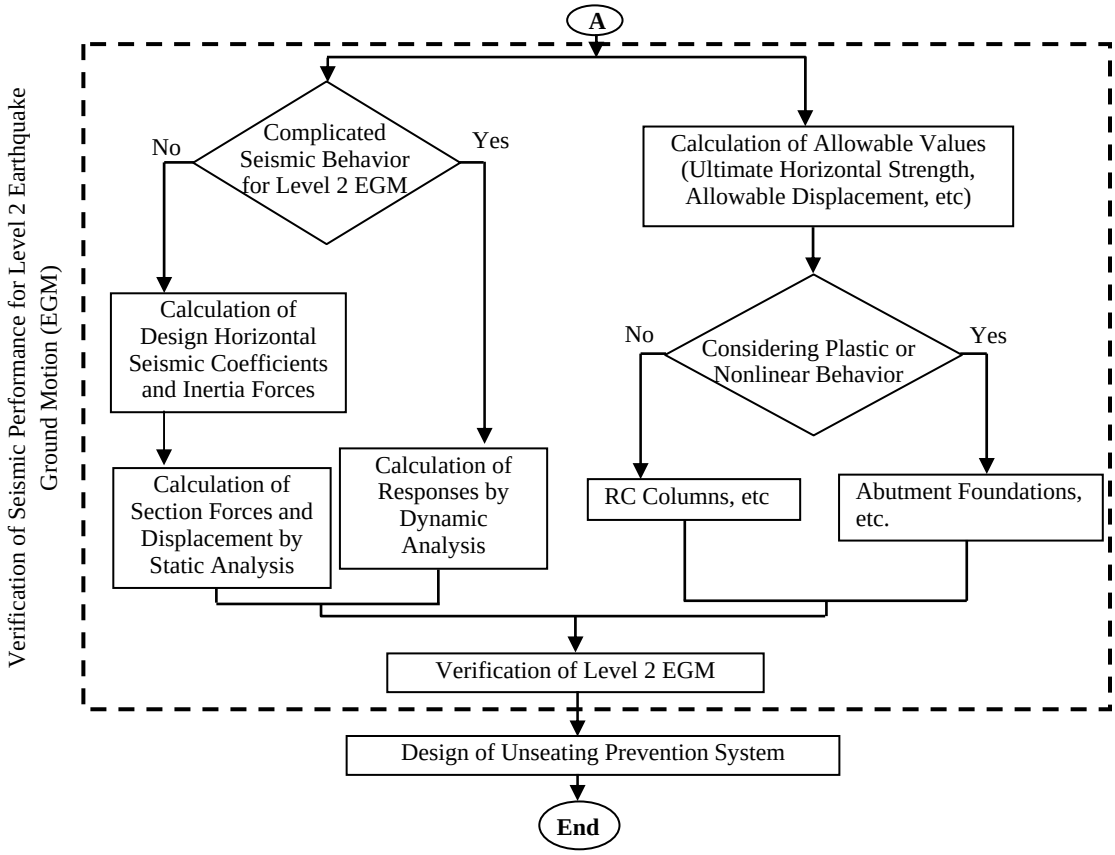
Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not required	Not required
B, C or D	Equivalent Static or Elastic Dynamic Analysis	Elastic Dynamic Analysis

- Nonlinear time history procedure is generally not required unless:
  - P-D effects are too large to be neglected,
  - Damping provided by a base isolation system is large, and
  - Requested by Owner.

JRA Part V – Seismic Design, 2012 Ed.

(1) The procedure for seismic design is illustrated below:





(2) The method for seismic analysis:

- Although dynamic analysis methods can be applied to bridges without complicated seismic behavior, it is recommended to use static analysis methods because the verification in accordance with static method is generally feasible for these bridges.
- Since the seismic behavior of bridges with predominant first mode of vibration and plural plastic behavior or bridges in which investigation on application of Energy Conservation Principle remains unclear may become complicated due to plasticity of members, their Seismic Performance Level 1should be verified by the static analysis methods but Seismic Performance Level 2 or Level 3 be verified by dynamic methods.

Relationship between Complexities of Seismic Behavior and Design Methods Applicable for Seismic Performance Verification

Dynamic characteristics of bridges / Seismic Performance to be verified	Bridges without complicated seismic behavior	Bridges with plastic behavior & yielded sections, and bridges not applicable for Energy Conservation Principle	Bridges of likely important higher modes	Bridges not applicable for the Static Analysis Method
Seismic Performance Level 1	Static analysis	Static analysis	Dynamic analysis	Dynamic analysis
Seismic Performance Level 2 & Level 3	Static analysis	Dynamic analysis	Dynamic analysis	Dynamic analysis
Examples of applicable bridges	Other than bridges shown in the right columns	- Bridges with rubber bearings to disperse seismic horizontal forces - Seismically isolated bridges - Rigid-frame bridges - Bridges with steel piers likely to generate plasticity	- Bridges with long natural period - Bridges with high piers	- Cable-type bridges such as cable-stayed bridges and suspension bridges - Deck-type & half through type arch bridges - Curved bridges

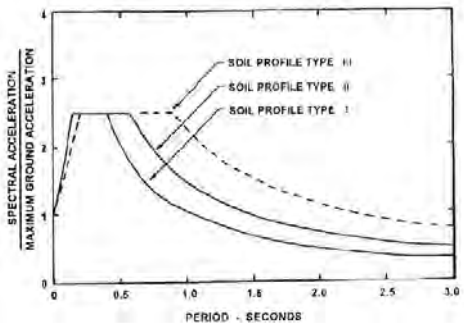
6.3.4 Acceleration Response Spectra

The design response spectrum used in the analysis to determine the demand forces and displacements are functions of soil types and site characteristics.

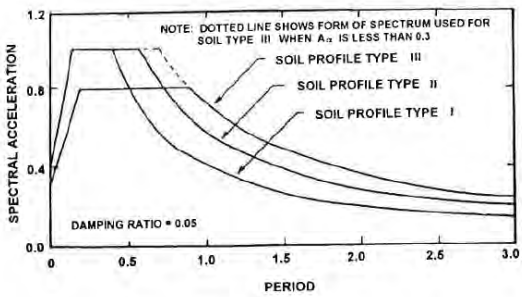
DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition

- The Standards assigns two (2) seismic zones in the Philippines as shown in the figure. Basically, the whole of the country belongs to Zone 4, except Palawan which is in Zone 2.
- The design ground motion spectra for 5% damping is developed for the 3 soil type conditions, as shown in the figure for normalized spectra and spectra with the effective peak acceleration (EPA) A=0.40.

Seismic Zone Map in the Philippines



(a) normalized



(b) for A=0.4

Design Response Spectra

AASHTO LRFD Bridge Design Specifications, 2012, 6th Ed.

- The general procedure to develop the design spectrum is to use the peak ground acceleration coefficient (PGA) and the short and long period spectral acceleration coefficients (S_S and S_1) based on the maps prepared in the specifications.
- The PGA and the spectral acceleration coefficient maps are developed based on the design earthquakes with 7 percent probability of exceedance in 75 years (approximately 1,000 return period.
- The five-percent-damped-design response spectrum shall be taken as specified in the figure below. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients scaled by zero-, short-, and long-period site factors, F_{pga}, F_a and F_v respectively.
- Each bridge is assigned to one of the four seismic zones in accordance with the table below using the value of S_{D1} given by the equation:

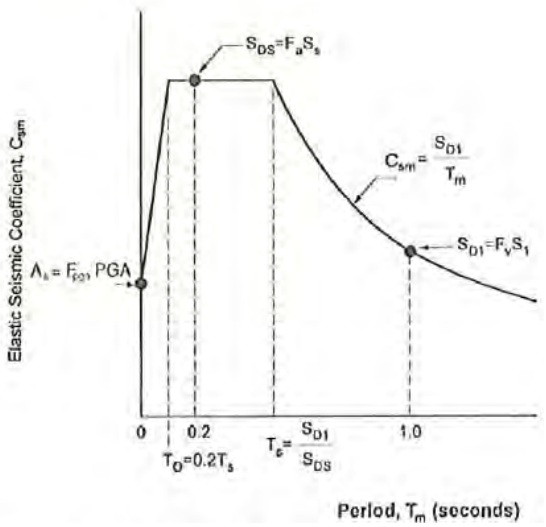
$S_{D1} = F_v S_1$

where: F_v = site factor for long-period range of acceleration response spectrum

S_1 = horizontal response acceleration coefficient at 1.0s period on rock (Site Class B)

Seismic Zones

Acceleration Coefficient, S_{D1}	**Seismic 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



AASHTO Design Response Spectrum

where:

A_s = peak seismic ground acceleration coefficient modified by short-period site factor

F_{pga} = site factor at zero-period on acceleration response spectrum

PGA = peak seismic ground acceleration coefficient on rock

S_{DS} = horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor

F_a = site factor for short-period range of acceleration response spectrum

S_s = horizontal response spectral acceleration coefficient at 0.2-s period on rock

C_{sm} = elastic seismic response coefficient for the mth mode of vibration

S_{D1} = horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor

F_v = site factor for long-period range of acceleration response spectrum

S_1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

- The general procedure to develop the design spectrum is to use the peak ground acceleration coefficient (PGA) and the short and long period spectral acceleration coefficients (S_S and S_1) based on the maps prepared in the specifications.
- The PGA and the spectral acceleration coefficient maps are developed based on the design earthquakes with 7 percent probability of exceedance in 75 years (approximately 1,000 return period.
- The five-percent-damped-design response spectrum shall be taken as specified in the figure below. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients scaled by zero-, short-, and long-period site factors, F_{pga}, F_a and F_v respectively.
- Each bridge is assigned to one of the four seismic design category (SDCs) in accordance with the table below based on S_{D1}:

$$A_s = F_{pga}\, PGA$$

$$S_{DS} = F_a\, S_s$$

$$S_{D1} = F_v\, S_1$$

Seismic Zones

Value of $S_{D1} = F_v\, S_1$	**SDC**
$S_{D1} \leq 0.15$	A
$0.15 < S_{D1} \leq 0.30$	B
$0.30 < S_{D1} \leq 0.50$	C
$0.50 < S_{D1}$	D

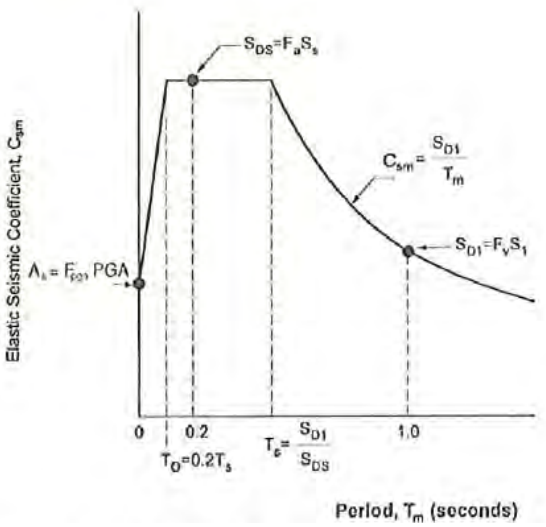
where: F_{pga} = site coefficient for peak ground acceleration coefficient

F_a = site coefficient for for 0.20s period spectral acceleration

F_v = site factor for 1.0s-period spectral acceleration

S_s = 0.2s period spectral acceleration coefficient on Class B rock

S_1 = 1.0s period spectral acceleration coefficient on Class B rock



AASHTO Design Response Spectrum
(*Construction Using Three=Point Method)*

where:

A_s	=	peak seismic ground acceleration coefficient modified by short-period site factor
F_{pga}	=	site factor at zero-period on acceleration response spectrum
PGA	=	peak seismic ground acceleration coefficient on rock
S_{DS}	=	horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor
F_a	=	site factor for short-period range of acceleration response spectrum
S_s	=	horizontal response spectral acceleration coefficient at 0.2-s period on rock
C_{sm}	=	elastic seismic response coefficient for the mth mode of vibration
S_{D1}	=	horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor
F_v	=	site factor for long-period range of acceleration response spectrum
S_1	=	horizontal response spectral acceleration coefficient at 1.0-s period on rock

For periods less than or equal to T_o, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = (S_{DS} - A_s)\, T/ T_o + A_s$$

For periods greater than of equal to T_o, and less than of equal to T_S, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = S_{DS}$$

For periods greater than T_S, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = S_{D1}/T$$

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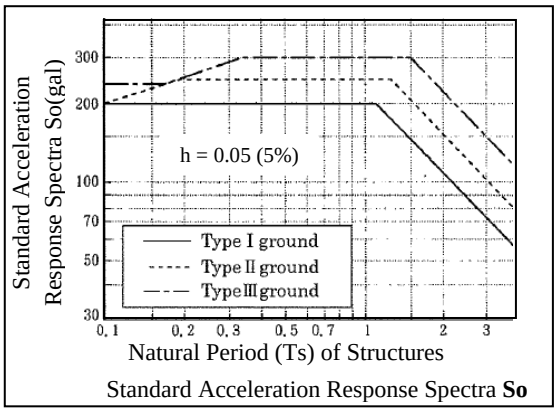
S = C_Z*C_D*$\mathbf{S_0}$
(S: ARS for Level 1EGM, S_0: SARS)

SI = C_Z*C_D*$\mathbf{S_{I0}}$
(SI: Type I ARS for Level 2 EGM, S_{I0}:SARS)

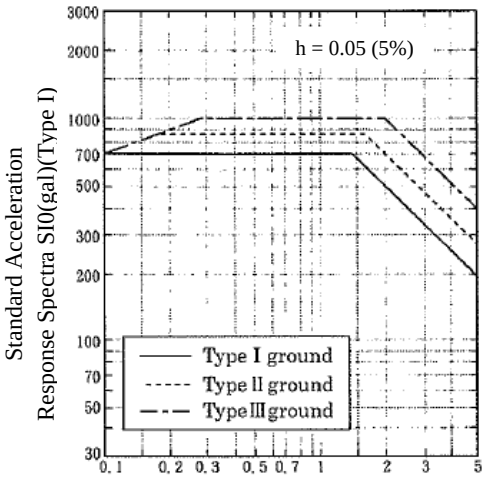
SII = C_Z*C_D*$\mathbf{S_{II0}}$
(SII: Type II ARS for Level 2 EGM, S_{II0}: SARS)
(SARS= **S**tandard **A**cceleration **R**esponse **S**pectra,
ARS= **A**cceleration **R**esponse **S**pectra,
EGM = **E**arthquake **G**round **M**otion)

C_D: Modification factor for damping ratio (h) of structures

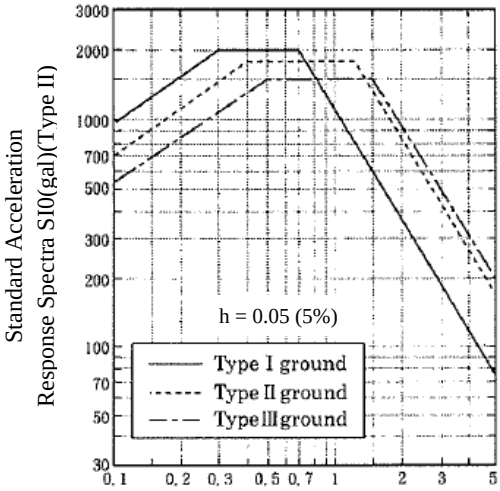
C_Z: Modification factor for zones



Level 1 Earthquake Ground Motion

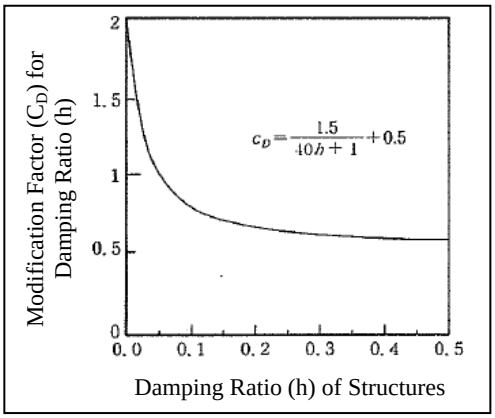


Standard Acceleration Response Spectra $\mathbf{S_{I0}}$ (Type I)



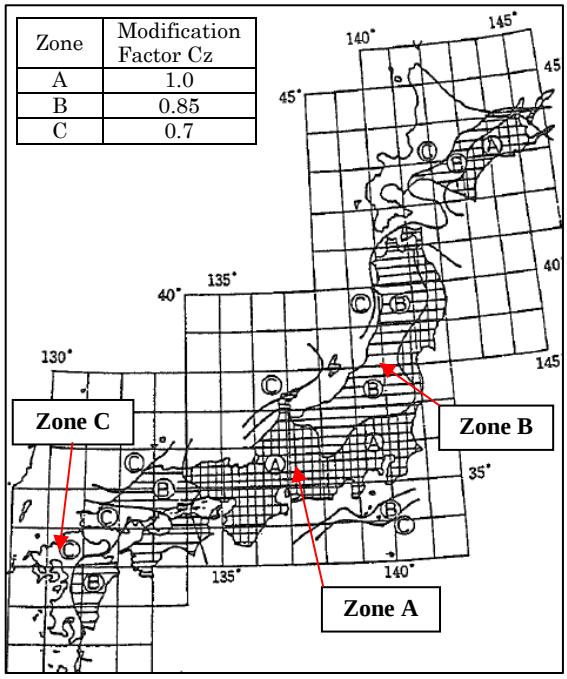
Standard Acceleration Response Spectra $\mathbf{S_{II0}}$ (Type II)

Level 2 Earthquake Ground Motions



Modification Factor (CD) for Damping Ratio (h) of Structures

Zone	Modification Factor Cz
A	1.0
B	0.85
C	0.7



Modification Factors for Zones, Cz

6.3.5 Unseating/Fall-Down Devices

DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition

1. Minimum Support Length Requirements

- Bearing seats supporting the expansion end of girders shall be designed to provide a minimum support length N (mm) not less than:

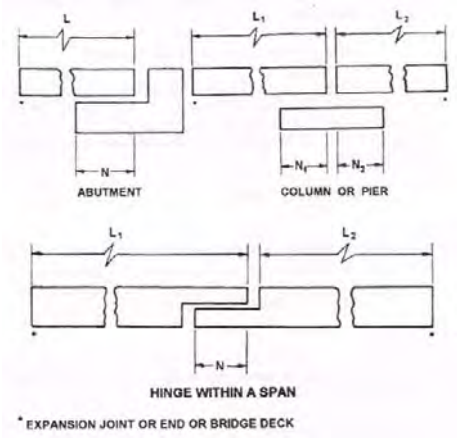
$$N = 305 + 2.5L + 10H$$

where:

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L_1 and L_2, the distances to either side of the hinge (m)

H = average height of columns supporting the bridge deck to the next expansion joint (m)

- For columns and Piers: H = column or pier height (m)
- For hinges within a span: H = average height of the adjacent two columns or piers (m)



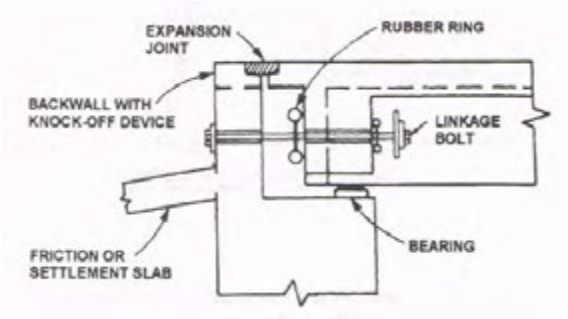
Dimension for minimum support length

2. Horizontal Linkage/Longitudinal Restrainers

- Positive horizontal linkage shall be provided between adjacent sections of the superstructure at supports and expansion joints within a span. The linkage shall be designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure.
- If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motion, sufficient slack must be allowed in the linkage so that the linkage force does not start to act until the design displacement is exceeded.
- Where linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans.
- Positive linkage shall be provided by ties, cables, dampers or equivalent mechanism. Friction shall not be considered a positive linkage.

3. Hold Down Device

- Hold down devices shall be provided at supports and at hinges in continuous structures, where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50%, but is less than 100% of the dead load reaction.
- If the vertical forces result in uplift, the hold down device shall be designed to resist the larger of the following net upward force:
  - 120% of the difference between the vertical seismic force (Q) due to longitudinal horizontal seismic load and the dead load reaction (DR), or
  - 10% of the dead load downward force that would be exerted if the span were simply supported.



Horizontal Linkage

AASHTO LRFD Bridge Design Specifications, 2012, 6th Ed.

1. Minimum Support Length Requirements

- Support lengths at expansion bearings without restrainers, shock transmission unit (STUs), or damper shall either accommodate the greater of the:
  - maximum displacement calculated in the inelastic dynamic response analysis
  - or a percentage of the empirical support length , N

  Otherwise, longitudinal restrainers shall be provided. Bearings restrained for longitudinal movements shall be design in accordance with the calculated seismic design forces.

- The empirical support length is shown in the equation below while the percentage of N applicable to each seismic zone in given in the table below:

$$N = (200 + 0.0017L + 0.0067H)(1 + 0.000125S^2)$$

Percentage N by Zone and Acceleration Coefficient

Zone	Acceleration Coefficient, A_S	Percent, N
1	<0.05	≥75
1	≥0.05	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

where:

N = minimum support length measured normal to the centerline of bearing (mm)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (mm)

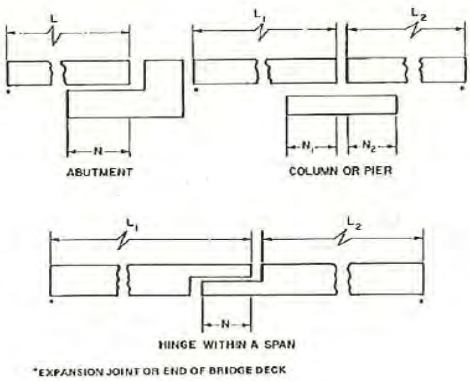
S = skew of support measured from line normal to span (°)

H = for abutments, average height of columns supporting the bridge deck to the next expansion joint (mm)

for columns and/or piers, column, or pier height (mm)

for hinges within a span, average height of the adjacent two columns or piers (mm)

0.0 for single-span bridges (mm)



Support Length, N

2. Longitudinal Restrainers

- Restrainers shall be designed for a force calculated as the acceleration coefficient, As, times the permanent load of the lighter of the two adjoining spans or parts of the structure.
- Sufficient slack shall be allowed in the restrainer so that the restrainer does not start to act until the design displacement is exceeded.

3. Hold Down Device

- Hold down devices shall be provided at supports and at hinges in continuous structures for Seismic Zones 2, 3 and 4 where the vertical seismic force due to the longitudinal seismic load opposes and exceeds 50%, but less than 100% of the reaction due to permanent loads.

- If the vertical forces result in uplift, the hold down device shall be designed to resist the larger of:
  - 120% of the difference between the vertical seismic force and the reaction due to permanent loads, or
  - 10% of the reaction due to permanent loads.

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

1. Seating Length (Minimum Support Length Requirements)

(a) Seismic Design Categories A, B, and C

- Support lengths at expansion bearings without restrainers, shock transmission unit (STUs), or damper shall either accommodate the greater of the:
  - maximum displacement calculated in the inelastic dynamic response analysis
  - or a percentage of the empirical support length , N
- The empirical support length is shown in the equation below while the percentage of N applicable to each seismic zone in given in the table below:

$$N = (200 + 0.0017L + 0.0067H)(1 + 0.000125S^2)$$

Percentage N by Zone and Acceleration Coefficient

Zone	Acceleration Coefficient, A_S	Percent, N
1	<0.05	≥75
1	≥0.05	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

where:

N = minimum support length measured normal to the centerline of bearing (mm),

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within the span, L shall be the sum of the distances to either side of the hinge; for single span bridges, L equals to the length of the bridge deck (mm),

H = for abutment, average height of columns supporting the bridge deck from the abutments to the next expansion joint (mm),

S = angle of skew of support measured from a line normal to span (°).

(b) Seismic Design Category D

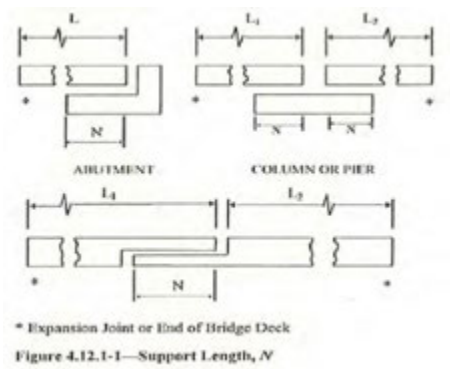
- For SDC D, hinge seat or support length, N, shall be available to accommodate the relative longitudinal earthquake displacement demand at the support or at the hinge within a span between two frames and shall be determined as:

$$N = (4 + 1.65\Delta_{eq})(1 + 0.00025S^2) \geq 24$$

where:

Δ_{eq} = seismic displacement demand of the long period frame on one side of the expansion joint (in),

S = angle of skew of support measured from a line normal to span (°).



Support Length, N

2. Longitudinal Restrainers

- Support restraints, used to achieve an enhanced performance of the expansion joint, may be provided for longitudinal linkage at expansion joints within the space and at adjacent sections of simply supported superstructures.
  - Friction shall not be considered to be an effective restrainer.
  - Restrainers shall be detailed to allow for easy inspection and replacement.
  - Restrainer layout shall be symmetrical about the centerline of the superstructure.
  - Restrainer systems shall incorporate an adequate gap for service conditions.
  - Yield indicators may be used on cable restrainers to facilitate post-earthquake investigations.

3. Superstructure Shear Keys

- For slender bents, shear keys on top of the bent cap may function elastically at the design hazard level.
- In lieu of experimental test data, the overstrength shear key capacity, V_{ok}, is taken as:

$$V_{ok} = 1.5V_n$$

where:

V_{ok} = overstrength shear key capacity used in assessing the load path to adjacent capacity-protected members (kip)

V_n = nominal interface shear capacity of shear key using nominal material properties and interface surface conditions (kip)

- For shear keys at intermediate hinges within a span, the designer shall assess the possibility of shear key fusing mechanism, which is highly dependent on out-of-phase frame movements.

JRA Part V – Seismic Design, 2012 Ed.

1. Seating Length

(1) Ordinary Bridge

- The equations below show the required seating length of a girder at its support.
- The seat length shall be measured in the direction perpendicular to the front line of the bearing support when the direction of soil pressure acting on the substructure differs from the bridge axis, as in case of askew bridge or a curved bridge.

$$S_E = U_R + U_G \geq S_{EM}$$

$$S_{EM} = 0.7 + 0.005 * Ls$$

$$U_G = \varepsilon_G * L$$

where,

S_E : Seat length

U_R : Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 EGM (m) (refer to description of U_R below)

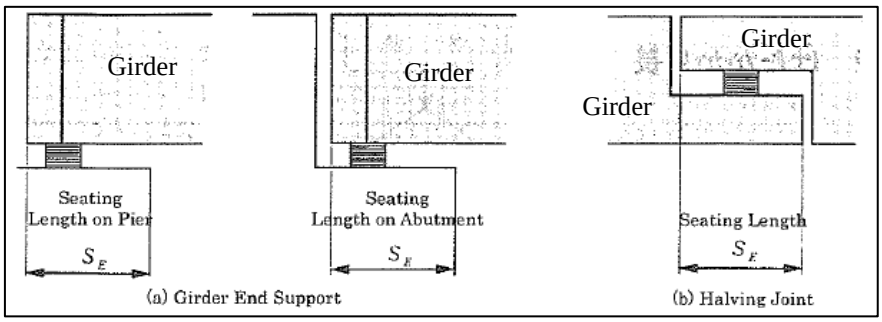
U_G : Relative displacement of the ground caused by seismic ground strain (m)

S_{EM} : Minimum seating length of a girder at the support

ε_G : Seismic ground strain
= 0.0025 for Ground Type I, 0.00375 for Ground Type II, 0.005 for Ground Type III

L : Distance between two substructures for determining the seating length (refer to description of L below)

Ls : Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer one shall be used.



Seating Length (S_E) (m)

(2) Skew Bridge

- The seating length shall be calculated by the following equation:

$$S_{E\theta} \geqq (L_\theta / 2)(\sin\theta - \sin(\theta - \alpha_E))$$

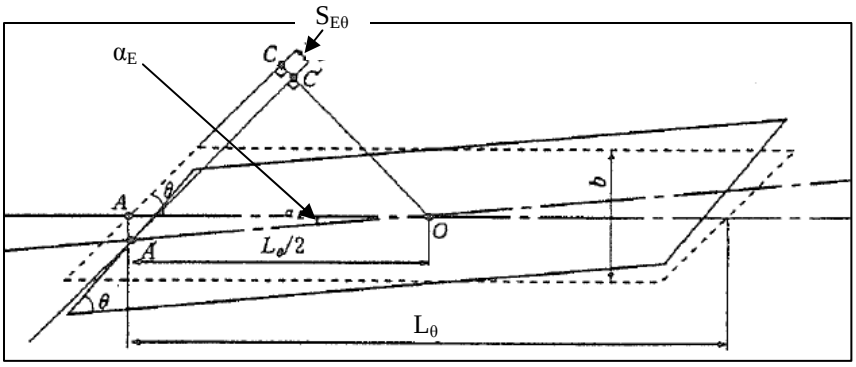
where

$S_{E\theta}$: Seating length for the skew bridge (m)

L_θ : Length of a continuous superstructure (m)

θ : Skew angle (degree)

α_E : Marginal unseating rotation angle (degree). α_E can generally be taken as 5 degrees.



Seating Length of a Skew Bridge

(3) Curved Bridge

- The seating length is to be calculated by:

$$S_{E\phi} \geqq \delta_E \frac{\sin\phi}{\cos(\phi/2)} + 0.3$$

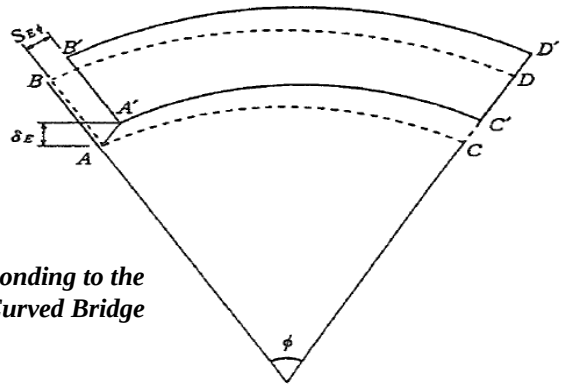
$$\delta_E = 0.005\,\phi + 0.7$$

where

$S_{E\phi}$: Seating length for the curved bridge (m)

δ_E : Displacement of the superstructure toward the outside direction of the curve (m)

ϕ : Fan-shaped angle by the two edges of a continuous girder of a curved bridge (degrees)



Seating Length Corresponding to the Movement of a Curved Bridge

2. Unseating Prevention Structure

(1) Ultimate Strength of an Unseating Prevention Device

- Ultimate strength of an unseating prevention device is to be calculated from the equation below.
- The unseating prevention structure is a structure that is 1) connecting the superstructure and the substructure, 2) providing protuberance either in superstructure and in the substructure, and 3) joining two superstructures together.

$$H_F = 1.5\,R_d$$

$$S_F = c_F S_E$$

where

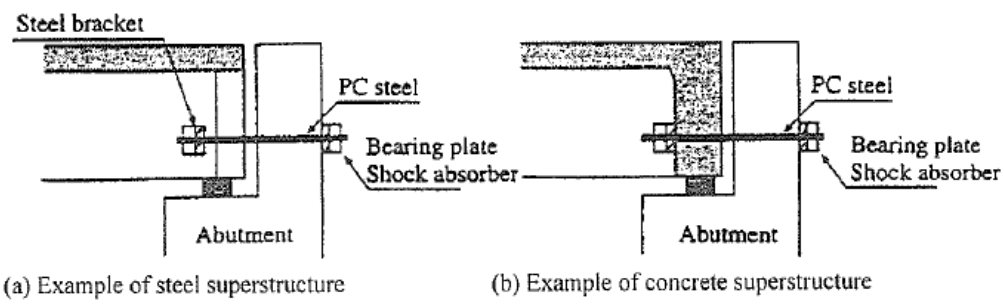
H_F : Design seismic force of the unseating prevention structure (kN)

R_d : Dead load reaction (kN). In case the structure connects two adjacent girders, the larger reaction shall be taken.

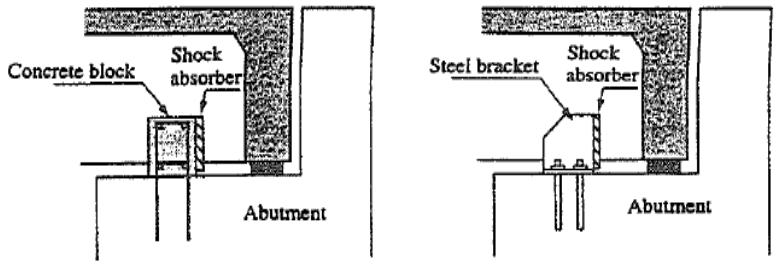
S_F : Maximum design allowance length of the unseating prevention structure (m)

S_E : Seating length

c_F :Design displacement coefficient of the unseating prevention structure. *(C_F = 0.75)*

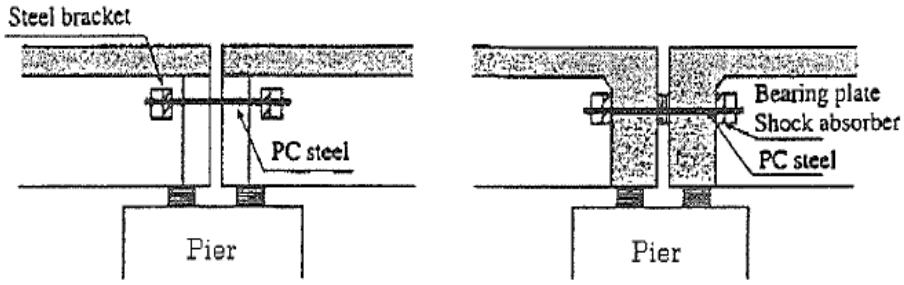


Unseating Prevention Structures Connecting the Superstructure with the Substructure



(a) Example of Concrete Block (b) Example of Steel Bracket

Unseating Prevention Structures Providing Protuberance on the Superstructure or the Substructure



(a) Example of Steel Superstructure (b) Example of Concrete Superstructure

Unseating Prevention Structures Connecting the Two Adjacent Superstructures

6.3.6 Foundation Design

DPWH D.O.75/NSCP Volume 2 – Bridges, 2005 Reprint of 2nd Edition

1. Foundation Analysis and Design
   - Foundation condition at the base of the columns and at the abutments may be modeled using equivalent linear spring coefficients.
   - The design forces for foundations including footings, pile caps and piles may be taken as either the forces determined from an elastic linear seismic analysis or the forces at the bottom of the columns corresponding to column hinging, whichever is smaller.
   - Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation supporting medium is used with the load combinations. Due consideration is given to the magnitude of the seismically-induced foundation settlement that the bridge can withstand.
   - Transient foundation uplift or rocking involving separation from the subsoil of up to one-half of an end bearing foundation pile group or up to one-half of the contact area of the foundation footings is permitted under seismic loading, provided that foundations are not susceptible to loss of strength under the imposed cyclic loading.
   - For saturated sand and soft clay foundation soils, due consideration shall be given to the potential for soil strength loss under the imposed cyclic loading in assessing the ultimate capacity of the foundation.
2. Pile Requirements
   - Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined based on the geotechnical investigation. Note that the ultimate capacity of the piles should be used in designing for seismic loads.

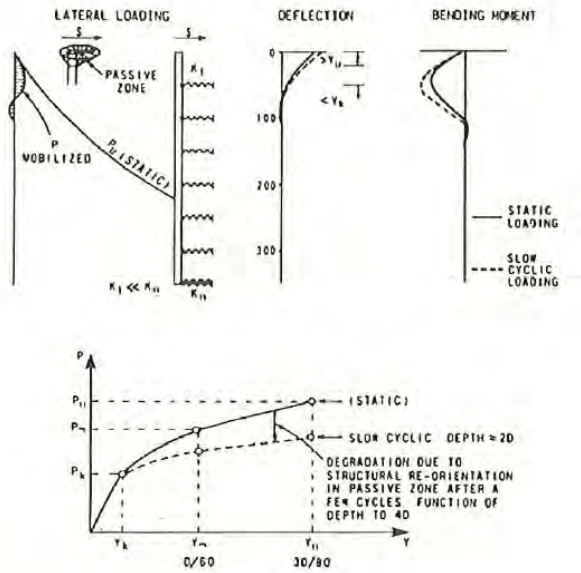
- Piles shall be adequately anchored to the pile footing or cap to develop uplift forces.
- Piles shall be detailed accordingly, including anchorage length of reinforcement, confinement length at potential plastic hinge region, volumetric ratio for confinement and minimum steel ratios.

3. Ground Liquefaction

- Where possible, the best design is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Calculations for lateral resistance would assume zero support from the upper zone of potential liquefaction.
- A philosophy of bridges in liquefaction susceptible areas might be one of "calculated risk", at least for those bridges regarded as being less essential for communication purposes immediately after an earthquake. However, design may be optimized so that the cost of repairs of potential earthquake damage to these bridges does not exceed the cost of remedial measures and additional construction needed to avoid the damage.
- Two basic approaches to evaluate cyclic liquefaction potential of saturated sand deposit subjected to earthquake loading are:
  - Empirical methods based on field observations of the performance of sand deposits in previous earthquake, and correlations between sites which have and have not liquefied and the relative density of the standard penetration test (SPT) blowcounts.
  - Analytical methods based on the laboratory determination of the liquefaction strength characteristics of undisturbed samples and the use of dynamic site response analysis to determine the magnitude of earthquake-induced shearing stresses.

4. Lateral Loading of Piles

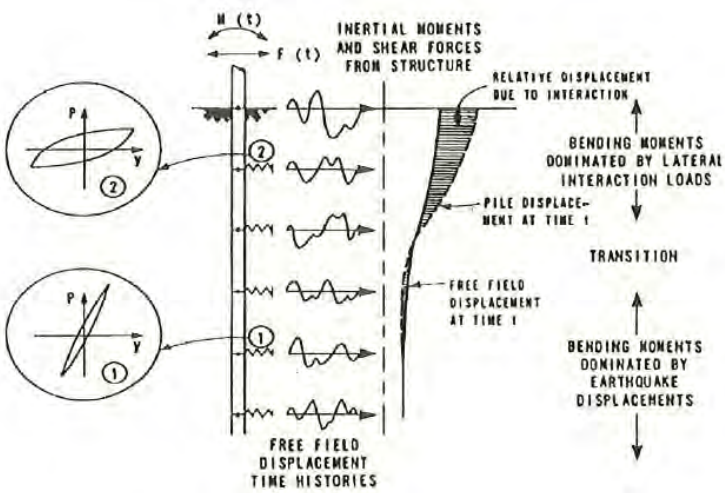
- A method introduced by the American Petroleum Institute (API) to calculate the lateral resistance of piles allowing soil failure utilizes a non-linear subgrade reaction of *P-y* curves for sands and clays. This method recognizes the degradation in lateral resistance with cyclic loading, although in the case of saturated sands, the degradation postulated does not reflect the pore water pressure increase.
- Under large loads, a passive failure zone develops near the pile head. Test data indicate that the ultimate resistance, p_u, for lateral loading is reached for pile deflections, y_u, of about 3d/80, where d is the pile diameter. Note that most of the lateral resistance is mobilized over a depth of 5d.



Lateral Loading of Piles in Sand using API Criteria
(AASHTO)

5. Soil-Pile Interaction

- The use of pile stiffness characteristics to determine the earthquake-induced pile bending moments based on a pseudo-static approach assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles and that the free-filed displacements themselves can influence bending moments.
- The free-field earthquake displacement time histories provide input into the lateral resistance interface elements, which in turn transfer motion into the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure.
- At greater depth (e.g. greater than 10d), where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a manner similar to that of the free-field, and pile bending moments become a function of the curvatures induced by free-field displacements.



Mechanism of Soil-Pile Interaction during Seismic Loading
(AASHTO)

AASHTO LRFD Bridge Design Specifications, 2012, 6th Ed.

1. Basic Policy

- Design of foundations at extreme limit states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

2. Mathematical Modeling of Foundation

- Mathematical models of bridge shall be based on the limit state being investigated and, where appropriate, including response characteristics of foundation.
- Appropriate representation of the soil and/or rock that supports the bridge shall be included in the mathematical model of the foundation. Moreover, for seismic design, gross soil movement and liquefaction should be considered.

3. Foundation Design

- The design forces for foundations including footings, pile caps and piles may be taken as either the forces determined for the extreme event load combination with seismic loads or the forces at the bottom of the columns corresponding to column plastic hinging.
- Extreme limit state design checks for spread footings shall include bearing resistance, eccentric load limitations (overturning), sliding, and overall stability.

- For seismic design, all soil within and above the liquefiable zone shall not be considered to contribute to the bearing resistance. Downdrag resulting from liquefaction induced settlement shall be determined and included in the load applied to the foundation.
- The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the P-y curve soil parameters should be reduced to account for liquefaction.

4. Liquefaction Design Requirements

- Liquefaction assessment shall be conducted for Seismic Zones 3 and 4 (acceleration coefficient > 0.30) depending on the ground water and soil characteristics. The assessment shall consider the loss in strength in the liquefied layers and flow failures, lateral spreading and slope instability.
- Bridges shall be analyzed and designed on two configurations – nonliquefied and liquefied configurations.
- During large magnitude earthquakes, the bridge response evaluation should consider the simultaneous occurrence of:
  - inertial response of the bridge, and loss of ground response from liquefaction around the bridge foundations, and
  - predicted amounts of permanent lateral displacement of the soil.

AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Ed.

1. Foundation Modeling Method (FMM)

- The foundation modeling method defined in the table below should be used as appropriate, including the requirements for estimating foundation springs for spread footings, pile foundation, and the depth to fixity for drilled shafts.

Definition of Foundation Modeling Methods (FMMs)

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Rigid for Site Classes A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil springs based on *P-y* curves

- The design forces for foundations including footings, pile caps and piles may be taken as either the forces determined from an elastic linear seismic analysis or the forces associated with the overstrength plastic moment capacity of the column or wall, whichever is smaller.

2. Spread Foundation

- For spread foundation, the spring constants shall account for the likely modes of seismic response (i.e. translation and rotation), the embedment and shape of the footing, the stiffness of the soil beneath the footing, and the effects of seismic loading on the stiffness of the soil.
- Spread footings shall not be used at locations where liquefaction or significant strength loss could occur during earthquake loading, unless the footing is located below the maximum depth of liquefaction or strength loss, or the ground has been improved such that liquefaction or strength loss will not occur.
- Footings satisfying the requirements below may be assumed to behave as rigid members

$$\frac{L-D_c}{2H_f} \leqq 2.5$$

where:
L = length of footing measured in the direction of loading
D_c = column diameter or depth in direction of loading
H_f = depth of footing

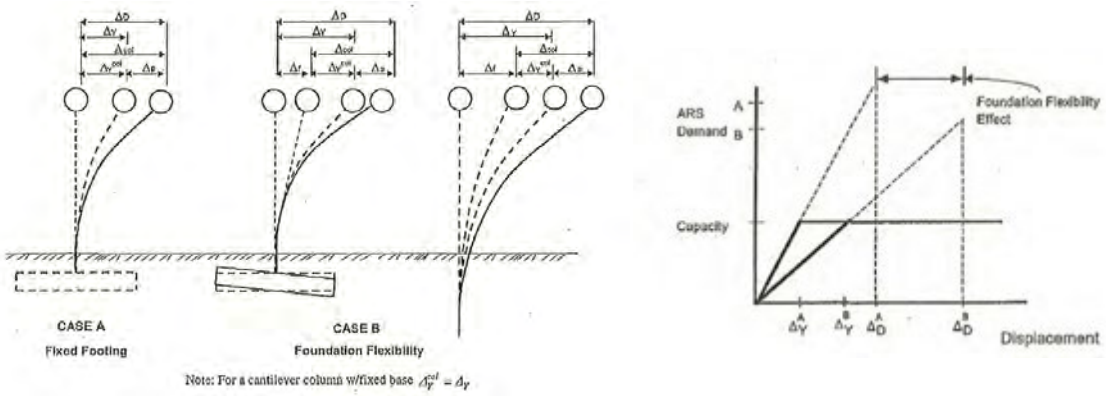
- The location of the resultant reaction forces shall be within the middle two-thirds of the base, if no live load is present. If full live load is present, the resultant shall be within the middle eight-tenths of the base.

3. Pile Foundation

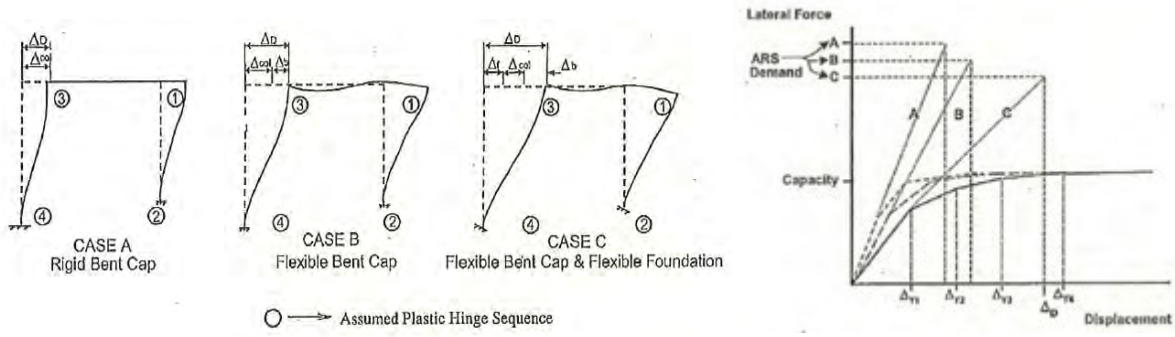
- For pile foundations, the structural and geotechnical elements of the foundation shall be taken into account, including the foundation flexibility. The nonlinear properties of the piles shall be considered in evaluating the lateral response of the piles to lateral loads during a seismic event. Liquefaction shall be considered during the development of spring constants and capacity values.
- The flexibility of drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis.

4. Effects of Foundation Flexibility

- The displacement demand for SDCs B, C, and D may be conservatively taken as the bent displacement inclusive of flexibility contribution from foundations, superstructure, or both.



Effects of Foundation Flexibility on the Force-Deflection Relation for a Single Column Bent



Effects of Foundation and Bent Cap Flexibilities on the Force-Deflection Relation for a Bent Frame

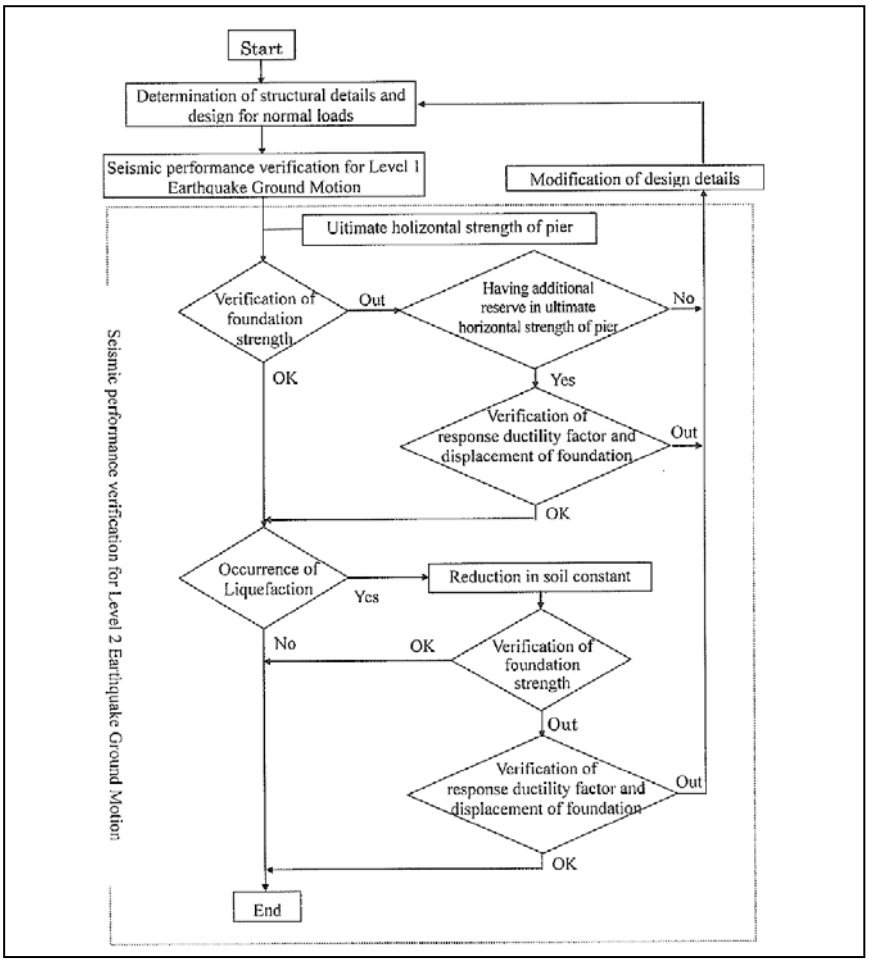
1. Verification Items

- The seismic performance, the limit state of the pier foundation and relationship with detailed verification items in verifying the limit state is presented below

Verification Items in Verification of Pier Foundation

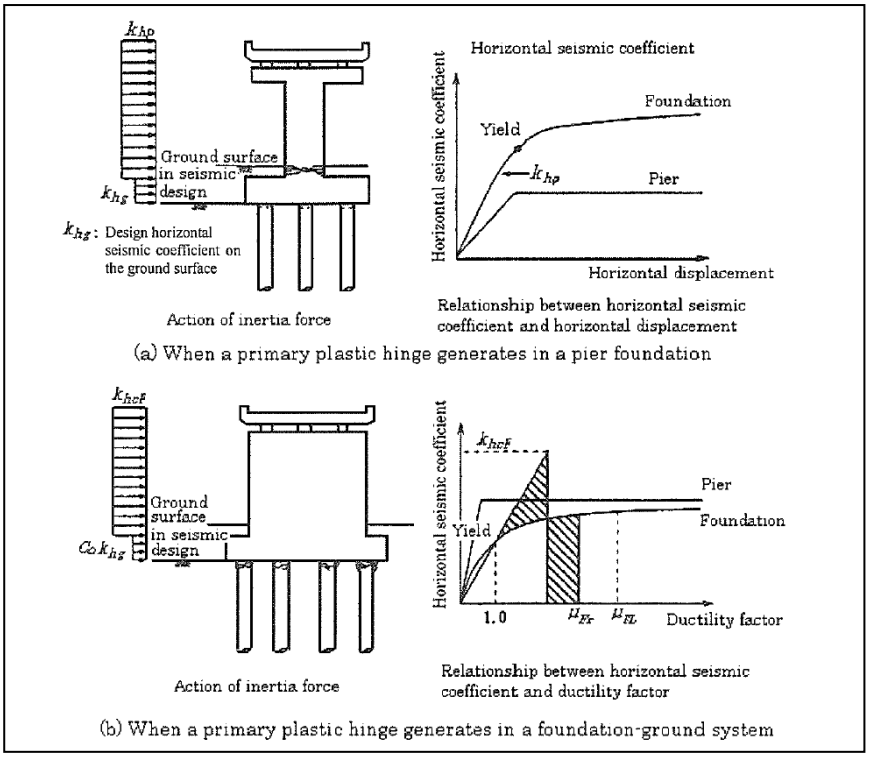
Limit State	Performance in Seismic Design	Verification Items	
With secondary plasticity only	Repairability in seismic design Serviceability in seismic design	About stability and repairability of main structure	· Foundation does not yield. · Sectional forces in foundation members are not greater than their strength.
		About excessive and residual displacement	· Satisfied by verifying the verification items about stability and repairability.
	Safety in seismic design	About loss of stability	· Satisfied by verifying the verification items about repairability and serviceability in seismic design.
		About excessive displacement	
Without excessive deformation or damage to interfere with recovery works	Repairability in seismic design Serviceability in seismic design	About stability and repairability of main structure	·Response ductility factor is less than ductility capacity · Sectional forces in foundation members are not greater than their strength.
		About excessive and residual displacement	· Response rotation at top section of the foundation is less than allowable rotation of the foundation.
	Safety in seismic design	About loss of stability	· Satisfied by verifying the verification items about repairability and serviceability in seismic design.
		About excessive displacement	

- The verification procedure for pier foundation by ductility design method is shown below:



Verification Procedure for Pier Foundation by Ductility Design Method

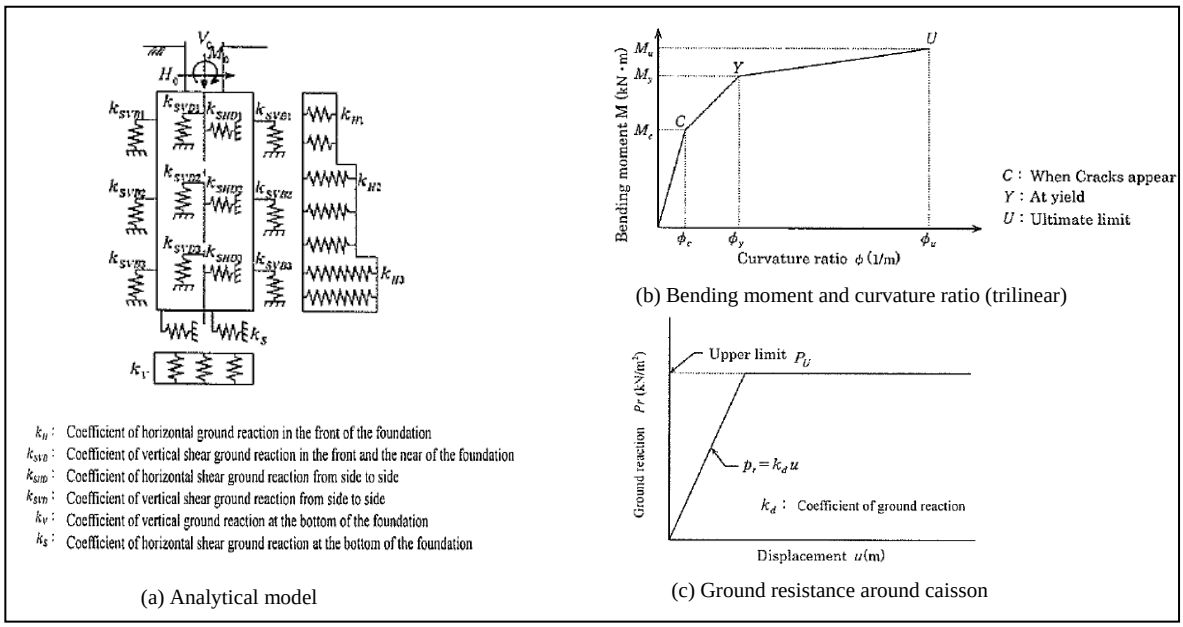
- Foundation shall be verified for the case without plastic behavior and the case with plastic behavior of the foundation (foundation yielding), as show below:



Seismic Design of Pier Foundation by Ductility Design Method

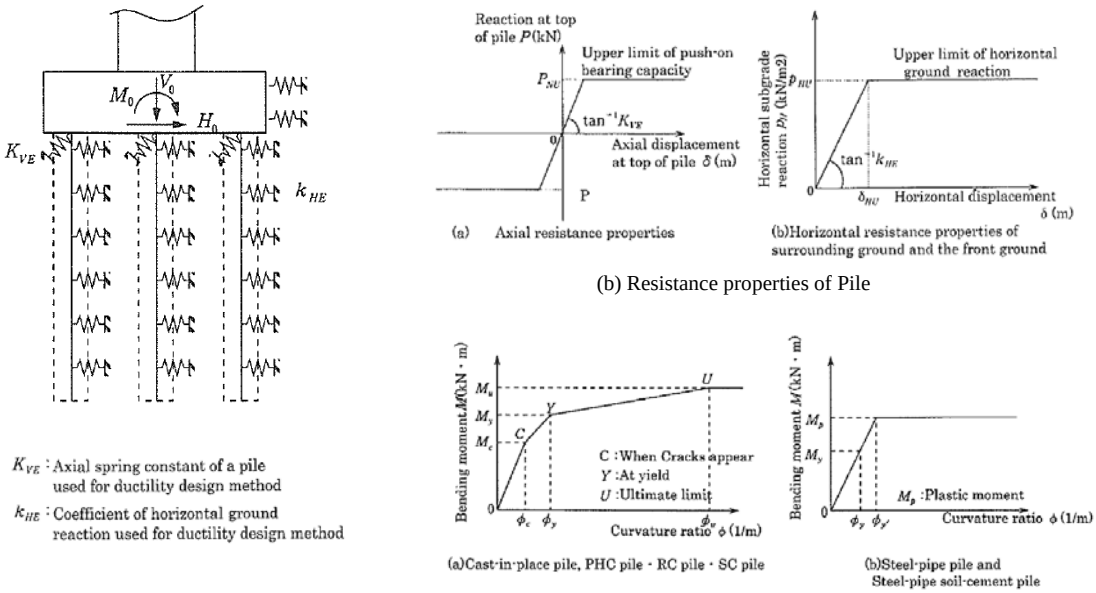
2. Sectional Force, Ground Reaction Force and Displacement of Pier Foundations

- When determining the behavior of a pier foundation subjected to seismic forces, the effects of ground resistance and nonlinearity of foundation shall be considered.
- <u>Caisson Foundation</u>. Caisson foundation shall be modeled into a single column considering the six ground reaction force resistance coefficients below:



Analytical Model of Caisson Foundation

- Pile Foundation. Pile foundation shall be modeled as a rigid-frame structure where the pile heads are rigidly connected to the pile cap.



(a) Analytical model

(b) Resistance properties of Pile

(c) Moment-curvature relationship

Analytical Model of Pile Foundation

3. Allowable Ductility Factor and Allowable Displacement of Pier Foundation
   - Allowable ductility factor and allowable displacement of pier foundation is determined by considering that the function of the bridge can easily be recovered, even though some failures may occur in the pier foundation.
4. Verification of Members of Pier Foundation
   - Sectional forces in members of pier foundation shall not exceed its ultimate strength.

6.3.7 Judgment of Liquefaction and its Consideration in Foundation Design

The following conditions in Table 6.3.7-1 require assessment of liquefaction potential at a site:

Table 6.3.7-1 Comparison between AASHTO and JRA Requirements for Site Liquefaction Potential Assessment

AASHTO	JRA
(1) Liquefaction Assessment For Seismic Zones 3 and 4, liquefaction assessment shall be conducted when both of the following conditions are present: ▪ ***Groundwater Level***. The groundwater level anticipated at the site is within 15.24m (50ft) of the existing ground surface or the final ground surface, whichever is lower. ▪ ***Soil Characteristics***. Low plasticity silts and sands within the upper 22.86m (75ft) are characterized by one of the following conditions: (a) the corrected standard penetration test (SPT) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and non-plastic silt layers, (b) the corrected cone penetration test (CPT) tip resistance, q_{ciN}, is less than or equal to 150 in sand and in non-plastic silt layers, (c) the normalized shear wave velocity, V_{s1}, is less than 660fps, or (d) a geologic unit is present at the site that has been observed to liquefy in past earthquakes.	**(1) Liquefaction Assessment** Sandy layer requiring liquefaction assessment: ▪ ***Groundwater Level***. Saturated soil layer having ground water level higher than 10m below the ground surface and lying at a depth less than 20m below the ground surface. ▪ ***Soil Characteristics***. (a) Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index Ip less than 15, even if FC is larger than 35%. (b) Soil layer having a mean particle size (D_{50}) less than 10mm and a particle size at 10% pass (on the grading curve) (D_{10}) is less than 1mm.
(2) Evaluation of Potential Effects of Liquefaction For sites that require assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction: ▪ Loss in strength in the liquefied layer or layers, ▪ Liquefaction-induced ground settlement, and ▪ Flow failures, lateral spreading, and slope instability.	**(2) Reduction of Geotechnical Parameters** ▪ For sandy layers with liquefaction potential, the geotechnical parameters in seismic design shall be reduced in accordance with the liquefaction resistance factor. ▪ The weight of the soil with reduced or zero geotechnical parameter is assumed to be acting as an overburden.

AASHTO	JRA
(3) Evaluation of Potential Effects of Liquefaction For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows: ▪ ***Non-liquefied Configuration***. The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a non-liquefied state. ▪ ***Liquefied Configuration.*** The structure as designed in non-liquefied configuration above shall be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified P-y curves, modulus of subgrade reaction, or t-z curves). The design spectrum shall be that used in a non-liquefied configuration.	**(3) Liquefaction-Induced Lateral Spreading** ▪ Ground with possible lateral movement include: (a) ground within 100m from water front in a shore formed by a revetment with 5m difference from ground surface to sea bottom, and (b) ground with sandy layer thicker than 5m that is assessed as a liquefiable layer in the area of a water front. ▪ Pier foundation situated in areas with possible lateral ground movement shall be verified against possible liquefaction-induced lateral spreading. In the verification, lateral movement force shall act on the pier foundation, and horizontal displacement at the top of the foundation shall not exceed two times the horizontal displacement at yielding of the foundation.

CHAPTER 7 IDENTIFICATION OF ISSUES ON CURRENT PRACTICE AND DPWH SEISMIC DESIGN SPECIFICATIONS FOR BRIDGES

7.1 General

In the Philippines the bridge seismic guidelines has been traditionally prepared based on the AASHTO design guidelines. However, geographical and geological characteristics including distribution of volcanoes, active faults and soft ground are largely different from those in U.S.A. If intending to secure the safety of structures, resulting in protecting nation's assets and people's life, against natural disasters, the country needs to adapt the guidelines to its local conditions, carefully identifying local particularities with general or universal ones.

The following two items may be key points in localization of the guidelines in terms of bridge seismic design, taking account of big differences in conditions between the Philippines and U.S.A.

- **Seismic hazard** (hazard sources related to seismic loads)
  - Active faults widely distributed in the Philippine islands,
  - Existence of several trenches sandwiching Philippine Archipelago such as Philippine Trench (the third deepest in the world), East Luzon Trench, Manila Trench, Negros Trench, and so on,
  - Distribution of active volcanoes.

- **Ground Conditions** (hazard sources related to both seismic loads and resistance to such)
  - Widely distributed relatively very soft layers,
  - Widely distributed sand or sandy soil layers having liquefaction potential.

The above two items may largely affect the intensity of an earthquake ground motion and structural damage patterns, leading to the extent of structural damages.

In addition to the above, current trend on the seismic design analysis method to assess the bridge seismic performance should be paid attention to , the design focus of which is being shifted from the force-based R-factor design approach to the displacement-based design approach after 1989 Loma Prieta earthquake, 1994 Northridge earthquake, 1995 Kobe earthquake, etc. Because neither the AASHTO force-based design specifications nor the LRFD design specifications provide detailed design criteria for estimating the ductility capacity of column subjected to the design earthquake. Estimation of a column's ductility capacity is essential for verification of the seismic performance requirements defined in the specifications.

In this chapter, the following five items are taken up as major issues on the seismic design specifications identified in this study, considering the above context.

- Necessity of Formulation of Policy on Seismic Performance Requirements.
- Establishment of Acceleration Response Spectra According to the Philippines' Geographical and Geological Characteristics.
- Issues on Soil Type Classification.
- Issues on Seismic Force Reduction Factor.
- Bridge Falling Down Prevention System.

7.2 Formulation of Policy on Seismic Performance Requirements

Seismic performance level and design seismic intensity should be considered as a set, namely, seismic performance levels are to be defined according to the design seismic intensity levels. The Philippines' bridge seismic specifications are developed entirely based on AASHTO specifications, which means that the Philippines' seismic performance requirements meet with AASHTO's ones regardless of their consciousness or intention if properly adapting AASHTO in their design.

AASHTO's seismic performance requirements are shown in Table 7.2-1.

Table 7.2-1 Seismic Performance Requirements by AASHTO LRFD

Earthquake Level	Bridge Category	Serviceability Performance	Safety Performance
Small /Moderate	All bridges	Should resist earthquake within the elastic range of the structural components	No significant damage
Large /Major	Critical bridges	Required to be open to all traffic once inspected after seismic event and usable by emergency vehicles for security, defense, economic or secondary life safety purposes immediately after the seismic event.	May suffer damage but with low probability.
	Essential bridges	Should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the seismic event and open to all traffic within days after the event.	
	Other bridges	May suffer significant damage and disruption to service	

Notes: with respect to Earthquake Level;

- Small/Moderate: not specified and no provision of the acceleration response spectrum.
- Large/Major: having a seven percent probability of exceedance in 75 years (return period 1,000 years), acceleration spectra by soil types provided by soil types.

JRA's seismic performance requirements are shown in Table 7.2-2 for reference. Comparing two tables, there is no significant differences in the requirements of seismic performance except for the following.

- JRA specifies seismic reparability in addition to serviceability and safety specified in both specifications.
- Acceleration response spectra for small/moderate earthquake level are not provided in AASHTO but provided in JRA.
- AASHTO defines the earthquake level as a return period, while JRA does not mention the return period because large or major earthquake levels are provided based on the highest ground motions recorded in the two past destructive earthquake types, a plate boundary type earthquake and an inland direct strike type earthquake.
- There are big difference in maximum response acceleration coefficients considered for seismic performance levels between AASHTO and JRA as follow.
    - AASHTO : 1.0 (soil type 1 & 2), 0.8 (soil type 3 & 4) (2007 version).
    - JRA : 2.0 (soil type I), 1.75 (soil type II), 1.5 (soil type III) (2012 version).
    - Both damping ratios are five percent.

Table 7.2-2 Seismic Performance Requirements by JRA (for reference)

Seismic Performance Level	Seismic Safety	Seismic Serviceability	Seismic Reparability	
			Emergency	Permanent
Performance Level 1	Prevent girders from unseating	Ensure the normal functions (within elastic limit states)	Need repair works to recover the functions.	Need only easy repair works
Performance Level 2	Ditto	Capable of recovering functions within a short period after the event.	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair works
Performance Level 3	Ditto	-	-	-

Earthquake Level			Bridge Category	
			Class B(high importance)	Class A(standard importance)
Seismic Level1 (small/moderate)	Highly probable during the bridge service life		Seismic Performance Level 1	
Seismic Level 2 (Large/Major)	Type I	A Plate Boundary Type Earthquake with Large Magnitude	Seismic Performance Level 2	Seismic Performance Level 3
	Type II	An Inland Direct Strike Type Earthquake		

Notes: with respect to Earthquake Level

- Acceleration response spectra for all seismic levels are provided, but the return periods are not mentioned.

:with respect to Bridge Category

- Class A and B are classified according to such importance factors as road class, bridge functions and structural characteristics.

Referring to the two tables above, the following tables (Table 7.2-3, Table 7.2-4 and Table 7.2-5) are recommended to be prepared or finalized by DPWH based on the proposals including options or study results to be submitted by JICA Study Team in the course of the Study, following AASHTO's policy on seismic performance requirements.

Table 7.2-3 Seismic Performance Requirements Corresponding to Earthquake Levels Shown in the Philippines

Earthquake Level	Bridge Category	Serviceability Performance	Safety Performance
Small /Moderate	All bridges	Should resist earthquake within the elastic range of the structural components	No significant damage
Large /Major	Critical bridges	To be formulated by DPWH	To be formulated by DPWH
	Essential bridges	To be formulated by DPWH	
	Other bridges	To be formulated by DPWH	

Table 7.2-4 Earthquake Levels

Earthquake Levels	Return Periods
Small / Moderate	To be determined by DPWH including no provision
Large / Major	To be determined by DPWH

Notes:
Considering the current practice in the Philippines or AASHTO, the following return periods are to be recommended or submitted for discussion and for convenience of DPWH's decision.

- Small/moderate: 100-year return period (about 50 % (or 75 %) probability of exceedance in 50 (75) years).
- Large/Major (1): 1000-year return period (about 7 % probability of exceedance in 75 years).
- Large/Major (2): 475-year return period which is currently used in the Philippines based on the previous AASHTO version
- Acceleration response spectra for three return periods above will be shown by means of probabilistic seismic hazard analysis (PSHA), taking account of the current information on distribution and scale of active faults and ocean trenches.

Table 7.2-5 Bridge Importance Category

Category	Definition
Critical Bridges	To be designated by DPWH
Essential Bridges	To be designated by DPWH
Other Bridges	To be designated by DPWH

Notes:
When bridge importance is classified in view of roles expected in the disaster prevention plan, if any, and road serviceability, the following should be considered (referring to JRA).

- To what extent a bridge is necessary for post-event and recovery activities as emergency transportation routes.
- To what extent damages to bridges (such as double-deck bridges and over-bridges) affect other structures and facilities.
- Present traffic volume of the bridge and availability of alternate bridge/route in case of the bridges losing pre-event functions.
- Difficulty (duration and cost) in recovering bridge function after the event.

As mentioned above, seismic performance requirements may be determined from due consideration of the relationship between the tolerable extent of damage and the scale of design acceleration response spectra (DARS). This, especially determination of the tolerable extent of damages according to the scale of DARS, is closely related to administrative decision, which is the Government's decision with consideration for such various situations of the country as its financial capacity and social and cultural acceptance about damages due to earthquakes.

7.3 Necessity of Establishment of Acceleration Response Spectra based on the Local Conditions

7.3.1 Development Methods of Acceleration Response Spectra for the Philippines

Current practice on acceleration response spectra is as follows.

- Seismic ground acceleration coefficients of 0.4 and 0.2 at the ground surface (may be soil type 1 or 2 specified in AASHTO).
- Since seismic coefficients at ground surface are deeply related to ground conditions as shown in Figure 7.3.1-1, the above expression is easy to cause misunderstanding that 0.4 or 0.2 can be used as the seismic coefficient at every ground conditions.

- Since seismic coefficients at soft ground surfaces are essential for assessment of liquefaction potential, these values should be clearly specified.
- Since the seismic coefficients of 0.4 and 0.2 are taken from parts of AASHTO seismic coefficient contour lines, active faults and ocean trenches in the Philippines are not taken into account of, together with ground conditions.

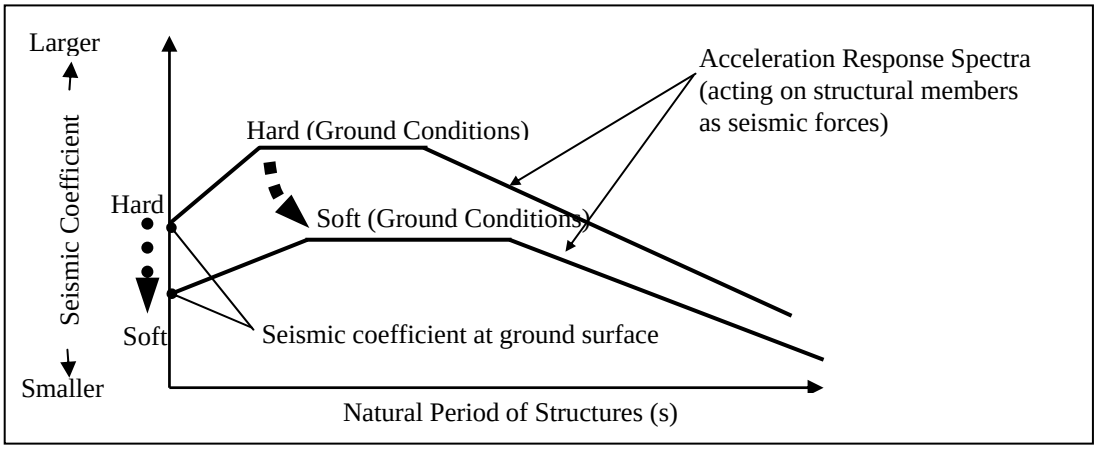


Figure 7.3.1-1 A Trend on Relationship between Seismic Forces and Ground Conditions

Considering the above, the following two analysis methods are recommended to formulate the seismic coefficients based on the Philippines' geographical and geological conditions, since no recorded data of strong earthquake ground motions is available.

Method 1 : Seismic response spectra for soft ground types in the Philippines will be formulated based on the procedure (concept) illustrated in Figure 7.3.1-2.

Method 2 : Seismic response spectra for all ground types in the Philippines will be formulated based on the procedure (concept) illustrated in Figure 7.3.1-3.

JRA method is shown in Figure 7.3.1-4 for reference since AASHTO method is almost the same as Method 2 above (USA has strong earthquake motions recorded, with which results obtained from Method 2 can be verified and modified or adjusted).

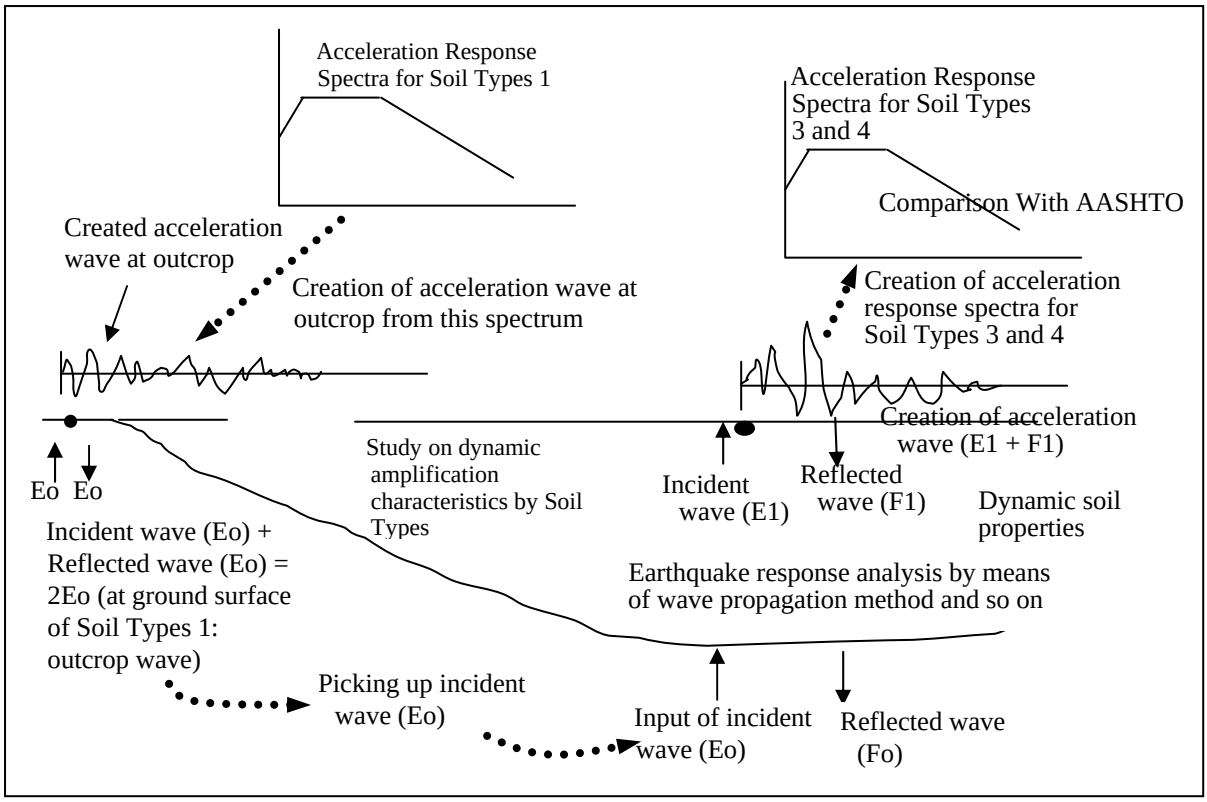


Figure 7.3.1-2 Study Procedure for Method 1

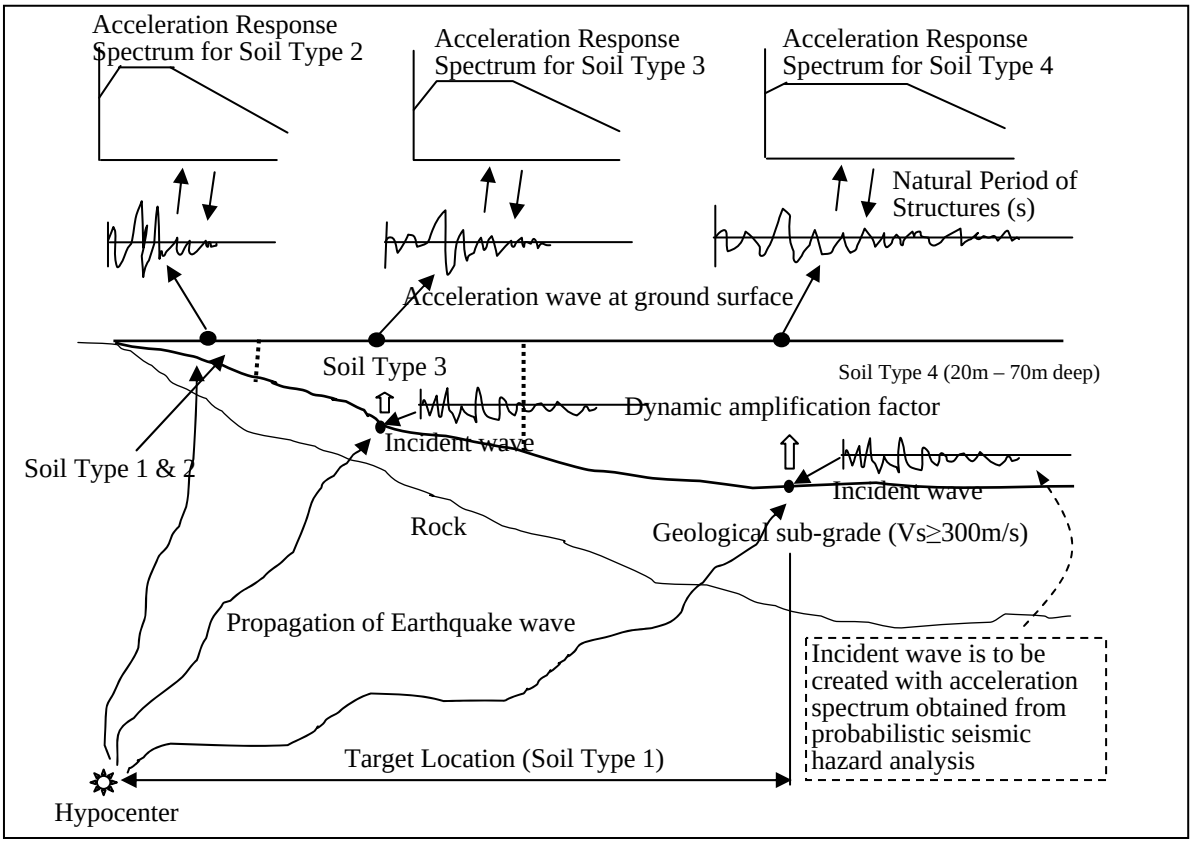


Figure 7.3.1-3 Study Procedure for Method 2

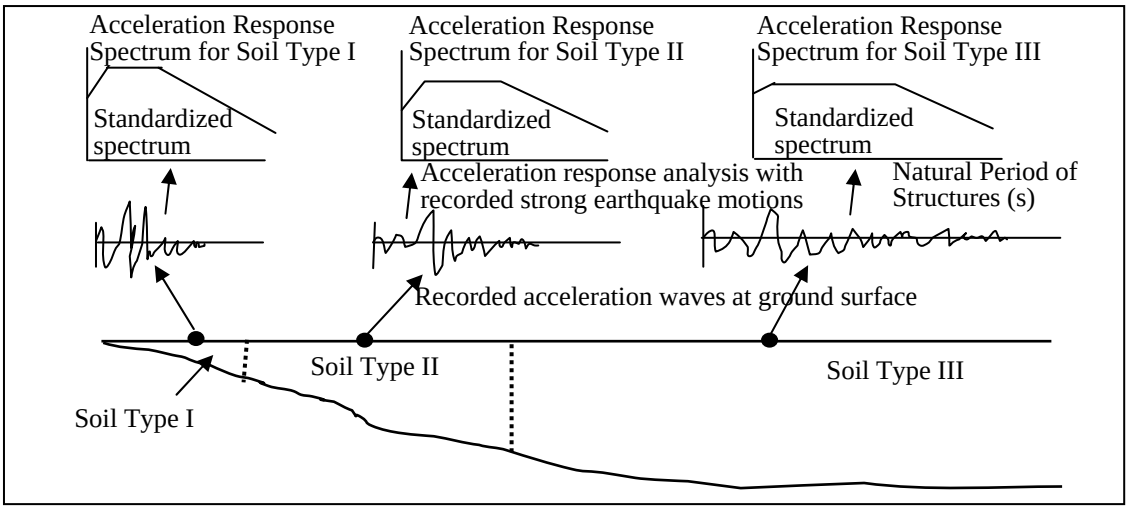


Figure 7.3.1-4 JRA Method (For Reference)

7.3.2 Recommendations

Taking account of current practice and localization of design seismic spectra, Method 2 is recommended. However, since design seismic spectra may directly affect the scale of structures, the following steps are to be taken for standardization, considering information on soil dynamic properties.

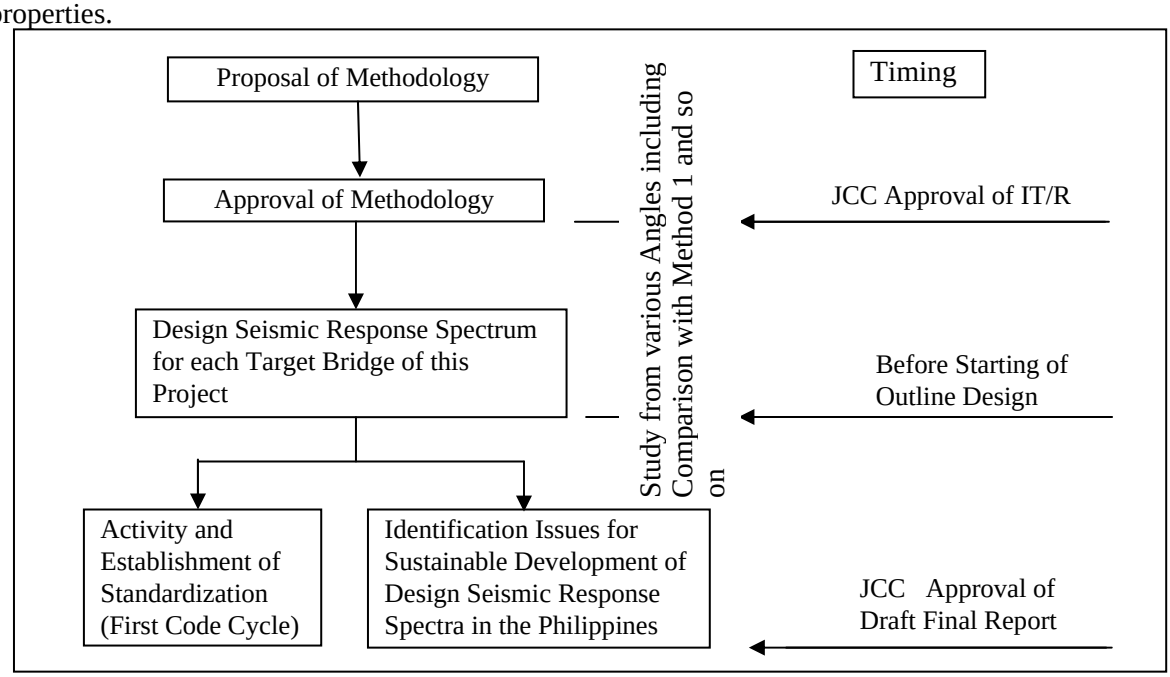


Figure 7.3.2-1 Flow of Establishment of Design Seismic Spectra

7.4 Ground Type Classification in Bridge Seismic Design

7.4.1 General

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 (AASHTO LFRD Bridge Design Specifications, 2012).

7.4.2 Soil Profile Type Classification under NSCP Vol.2 (2005)

The National Structural Code of the Philippines (Vol. 2 Bridges; 2nd Edition, 1997 and reprinted edition, 2005) defines three (3) types of soil profile in connection with the site classification (Table 7.4.2-1).

Table 7.4.2-1 Soil Profile Types of National Structural Code of the Philippines

Type	Soil Profile
I	- Rock of any characteristics, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 760 m/sec, or by other appropriate means of classification); or - Stiff soil conditions where the soil depth is less than 60 m. and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
II	Profile with stiff clay or deep cohesionless conditions where the soil depths exceeds 60 m. and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
III	Is a profile with soft to medium-stiff clays and sands, characterized by 10 m. or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.
	In location where the soils properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, the site coefficient for Soil Profile Type II shall be used. The soil profile coefficients apply to all foundation types including pile supported and spread footings.

7.4.3 Site Profile Types under AASHTO LFRD 2007

The AASHTO LFRD Bridge Design Specification (2007) defines four (4) types of soil profile in connection with the site classification (Table 7.4.3-1).
Currently DPWH uses the site profile types of AASHTO LFRD 2007.

Table 7.4.3-1 Soil profile types under AASHTO LFRD 2007

Type	Soil Profile
I	A profile shall be taken as Type I if composed of: - Rock of any description, either shale-like for crystalline in nature, or - Stiff soils where the soil depth is less than 60000 mm, and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays. (These materials may be characterized by a shear wave velocity greater than 765 m/sec.)
II	A profile with stiff cohesive or deep cohesionless soils where the soil depth exceeds 60 000 mm and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.
III	A profile with soft to medium-stiff clays and sands, characterized by 9000 mm or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils shall be taken as Type III.
IV	A profile with soft clays or silts greater than 12 000 mm in depth shall be taken as Type IV. (These materials may be characterized by a shear wave velocity of less than 152 m/sec. and might include loose natural deposits or manmade, nonengineered fill.)

7.4.4 Soil Profile Types under AASHTO LFRD 2012

The AASHTO LFRD Bridge Design Specification (2012) defines six (6) types of soil profile in connection with the site classification. Table 7.4.4-1 shows the definition of the soil profile types.

Table 7.4.4-1 Soil Profile Types under AASHTO LFRD 2012

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $[v_s]$>5,000 ft/s
B	Rock with 2,500 ft/sec<$[v_s]$<5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec<$[v_s]$<2,500 ft/s, or with either [N]>50 blows/ft, or $[S_u]$2.0 ksf
D	Stiff soil with 600 ft/s <$[v_s]$<1,200 ft/s, or with either 15<[N]50 blows/ft, or 1.0<$[S_u]$<2.0 ksf
E	Soil profile with $[v_s]$<600 ft/s or with either [N]<15 blows/ft or $[S_u]$<1.0 ksf, or any profile with more than 10 ft or soft clay defined as soil with PI>20, w>40 percent and $[S_u]$<0.5 ksf
F	Soils requiring site-specific evaluations, such as: ● Peats or highly organic clays (H>10 ft or peat or highly organic clay where H=thickness of soil) ● Very high plasticity clays (H>25 ft with PI>75) ● Very thick soft/medium stiff clays (H>120 ft)
Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data. Where: $[v_s]$ = average shear wave velocity for the upper 100 ft of the soil profile [N] = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile $[S_u]$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile PI = plasticity index (ASTM D4318) W = moisture content (ASTM D2216)	

Table 7.4.4-1 can be simplified and summarized as shown in Table 7.4.4-2.

Table 7.4.4-2 Simplified Soil Profile Types of AASHTO LFRD 2012

Site class	Rock, soil	(Vs): m/sec	(N)	(Su): ksf	Thickness (m)	PI	w (%)
A	Hard rock with	1,524 <					
B	Rock with	762 – 1,524					
C	- Very dense soil and soil rock with - Stiff soil with: or	366 – 762	50 <	2.0 <			
D	Stiff rock profile with:	183 – 366	15 – 50	1.0 – 2.0			
E	- Soil profile with - Soil profile with - Any profile with 3 m of soft clay with	< 183	15 <	< 1.0 < 0.5		20 <	40 <
F	- Organic or peat - Very high plasticity clays - Very thick soft/med. stiff clays				H > 3m H > 7.6m H > 36.6m		

7.4.5 Soil Profile Types under the Japan Road Association (JRA)

The Japan Road Association defines three (3) types of soil profile in connection with the site classification based on the characteristic values of the ground (T_G; Table 7.4.5-1).

Table 7.4.5-1 Soil Profile Type of JRA

Type	Soil Profile
I	T_G<0.2 (Very dense soils and/or rocks with Vs >=300 m/sec)
II	0.2<=T_G<0.6
III	0.6<=T_G

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

T_G: Characteristic value of the ground
n: Number of soil layers
H_i: A thickness (meter) of a soil layer (n=1,2,3…,i)
V_{si}: A shear wave velocity (meter/seconds) of a soil layer (n=1,2,3…,i)
A V_{si} is usually recalculated using the average N-value of the soil layer (i) as follows.

a) Sand/conhesionless soil ($1 \le N \le 50$) : $V_{si}=80\times N^{(1/3)}$ m/sec;
Silt/clay/cohesive soil ($1 \le N \le 50$) : $V_{si}=100\times N^{(1/3)}$ m/sec; or
b) N=0: V_{si}=50 m/sec.

7.4.6 Comparison of Soil Profile Types

The soil type classifications specified by ASEP, AASHTO and JRA can be comparable based on shear wave velocities as shown in Figure 7.4.6-1.

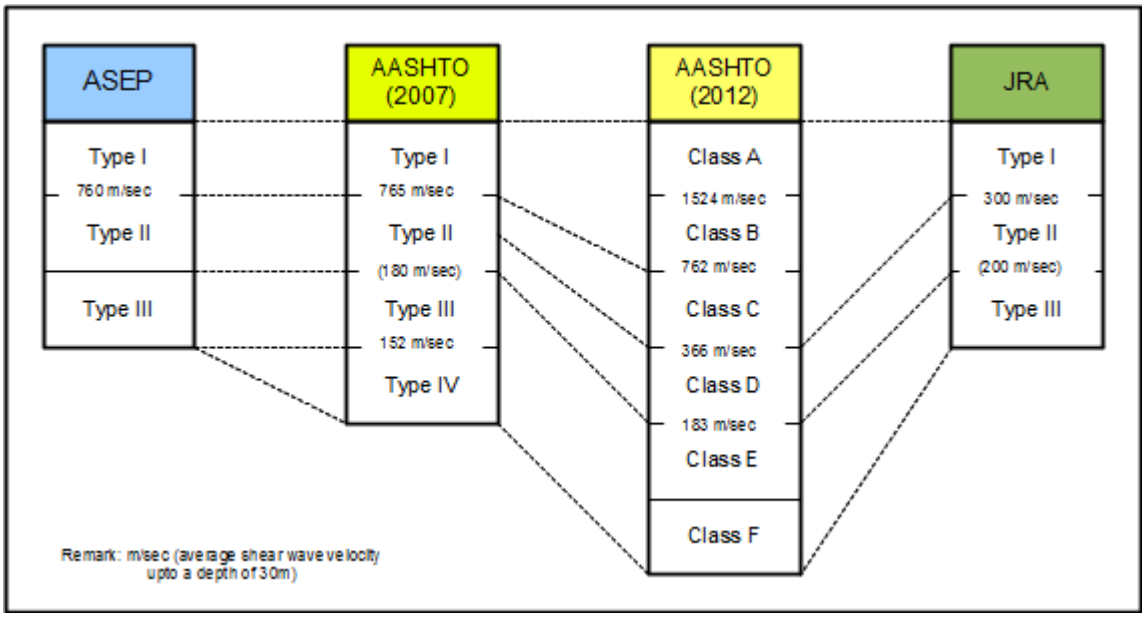


Figure 7.4.6-1 Comparison of Soil Profile Type Classification System

(1) ASEP and AASHTO (2007) /DPWH

The three (3) soil profile types defined by ASEP can be compared to the three (3) soil profile types (I, II, and III) defined in AASHTO (2007).
However in Philippines, delta, marsh, and shallow marine type soft alluvial sediments are considered to be present. It is also possible that soft soil layers characterized by a shear wave velocity of less than 150 m/sec (N<2) are distributed in the country. In this connection, a soil profile type like Type IV defined in AASHTO (2007) should be considered to apply for the seismic design for bridges.

(2) AASHTO (2007) and AASHTO (2012)

Definition of each soil profile types shown in AASHTO (2007) seems to be qualitative and requires "engineering judgments" to be given by skilled and well experienced specialists. In contrast, AASHTO (2012) shows a quantitative definition of the six (6) soil profile types. Each of the soil type classifications can be compared as shown in Figure 7.4.6-1.

(3) AASHTO (2012) and JRA

Type I of JRA is comparable with Classes A, B and C of AASHTO (2012). Type II of JRA is considered to be the same as Class D of AASHTO (2012). Type III of JRA is equivalent to Classes E and F of AASHTO (2012). AASHTO (2012) focuses on rocky sites more than JRA.

(4) Consideration on Site/Soil Profile Types

- The soil profile type classification defined by ASEP or AASHTO (2007) have qualitative and unclear definition. Methods of geological investigation are not clearly specified. Therefore, to determine the soil profile type for a site needs skilled engineers' judgment.
- AASHTO (2012) shows a more definitive and quantitative classification system. Each soil profile type can be identified using such as shear wave velocities or SPT blow counts. However in case of Class E, design ground motion spectra have to be determined site-specifically, and it probably needs additional geological studies, time, and cost. Moreover the soil profile types in AASHTO (2012) put emphasis on rocky sites. To determine one of three "rigid soil profile" from types A, B, or C, a shear wave velocity measurement test must be performed at a site.
- JRA's soil profile type classification system has quantitative and simple procedure. There are only three soil profile types but those covers all kinds of the ground condition. Shear wave velocities can be calculated from SPT blow counts (N-values) using simplified formulas defined in the specification. Therefore the soil profile types can be determined without in-situ shear wave measurement tests. N-values can be obtained in borings to confirm bearing layers, and it is economical and time-saving for the client.
- Philippines and Japan are classified into a kind of volcanic arc islands, and relatively unstable land geologically. In addition, Philippines and Japan has a similar tectonic setting. But, United States itself is continental crust and stable land basically. The geologic ages of rocks in Philippines and Japan are relatively younger than United States. On the other hand, United States of America are one of the oldest continents on the earth, and rock-prone country. In the United States, AASHTO has to cover the wide range of rock types rather than Philippines and Japan.

- It is considered that the soil profile type classification system of JRA is more suitable to Philippine than that of AASHTO, considering the considerations mentioned above.

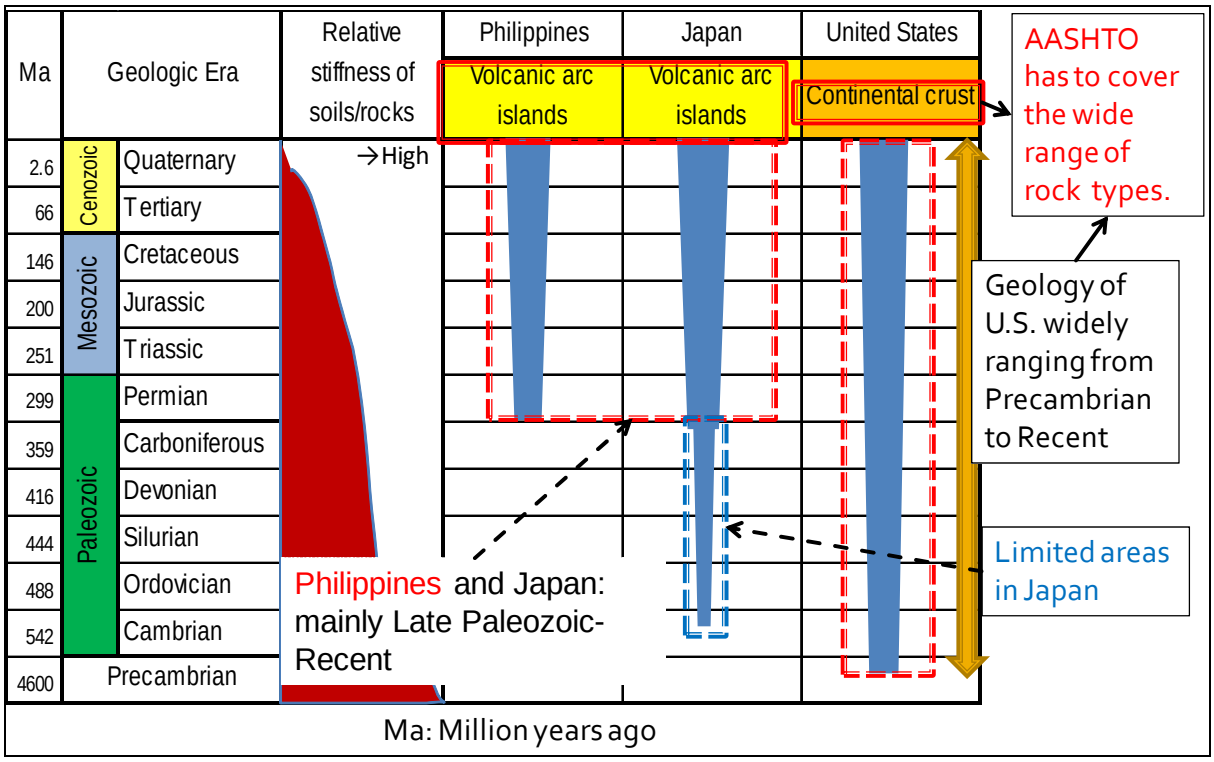


Figure 7.4.6-2 Geological Similarities/Difference among Three Countries (Philippines, Japan, and United States of America)

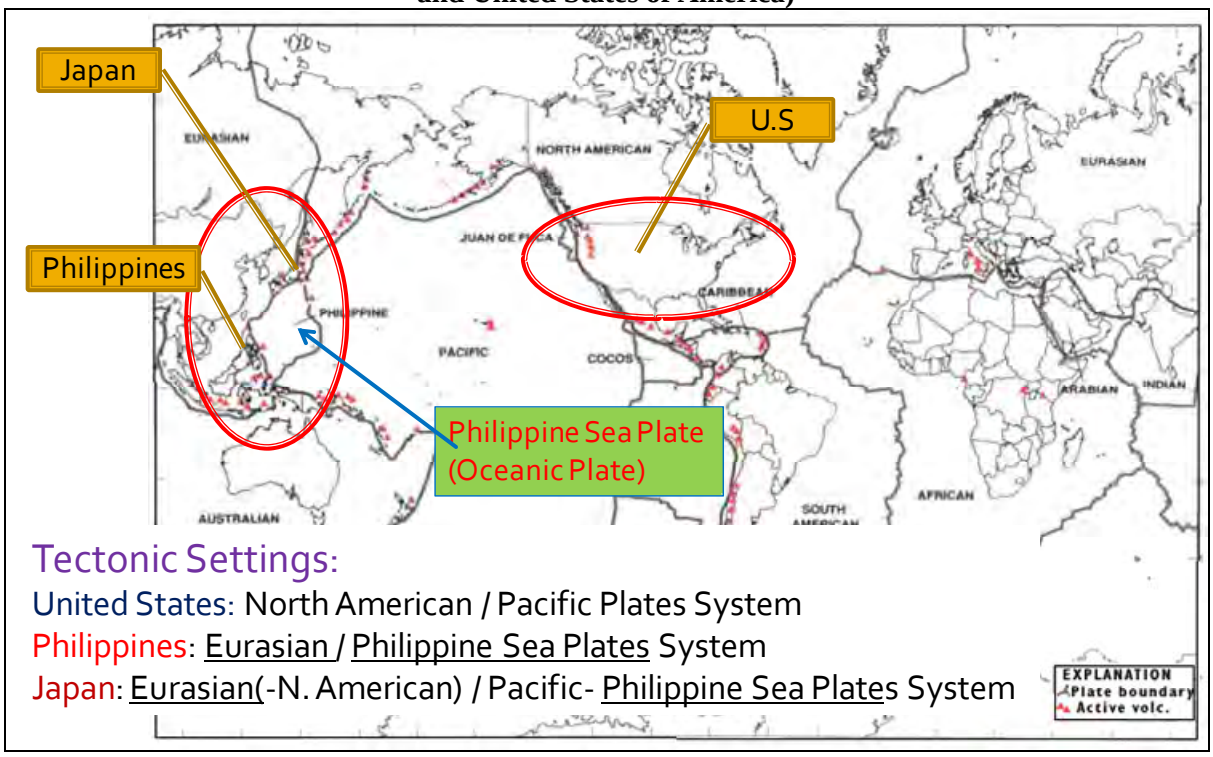


Figure 7.4.6-3 Tectonic Settings of Philippines, Japan, and United States of America

7.5 Issues on Seismic Response Modification Factor R

7.5.1 AASHTO Specifications for Response Modification Factor R

The AASHTO Specifications recognizes 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 force-reduction 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 the appropriate response modification factor, R (also referred to as the force-reduction factor). The R-factor basically represents the column ductility demand with consideration of ductility capacity, overstrength and structural redundancy. This R-factor is also used in the AASHTO 1996 edition.

Moreover, with the release of the *"AASHTO LRFD Bridge Design Specifications (1st Ed.)"* 1994, column ductility is likewise accounted for by using the response modification factor R. Values of the response modification factors, R, for the 1996 WSD/LFD and the 2012 LRFD AASHTO Specifications are presented in Table 7.5.1-1. Note that the 1996 R-factors are specified both in the NSCP 2005 and the DPWH Guidelines (2004 Draft).

The single values for the R-factors in the AASHTO 1996 edition was increased in the AASHTO LRFD 2012 edition (same as the 1994 edition) to three levels to account for the three bridge operational categories ("critical, "essential" and "other"), as indicated in Table 7.5.1-1.

Table 7.5.1-1 Response Modification Factors

(a) Response Modification Factors
(1996 AASHTO Standard Specifications for Highway Bridges (WSD/LFD))

Substructure	R
Wall-Type Pier	2
Reinforced Concrete Pile Bents	
1. Vertical Piles only	3
2. One or more Battered Piles	2
Single Columns	3
Steel or Composite Steel and Concrete Pile Bents	
1. Vertical Piles only	5
2. One or more Battered Piles	3
Multiple Column Bents	5

(b) Response Modification Factors - Substructures
(2012 AASHTO LRFD Bridge Design Specifications)

Substructure	Operational Category		
	Critical	Essential	Other
Wall-Type Pier	1.5	1.5	2.0
Reinforced Concrete Pile Bents			
1. Vertical Piles only	1.5	2.0	3.0
2. One or more Battered Piles	1.5	1.5	2.0
Single Columns	1.5	2.0	3.0
Steel or Composite Steel and Concrete Pile Bents			
1. Vertical Piles only	1.5	3.5	5.0
2. One or more Battered Piles	1.5	2.0	3.0
Multiple Column Bents	1.5	3.5	5.0

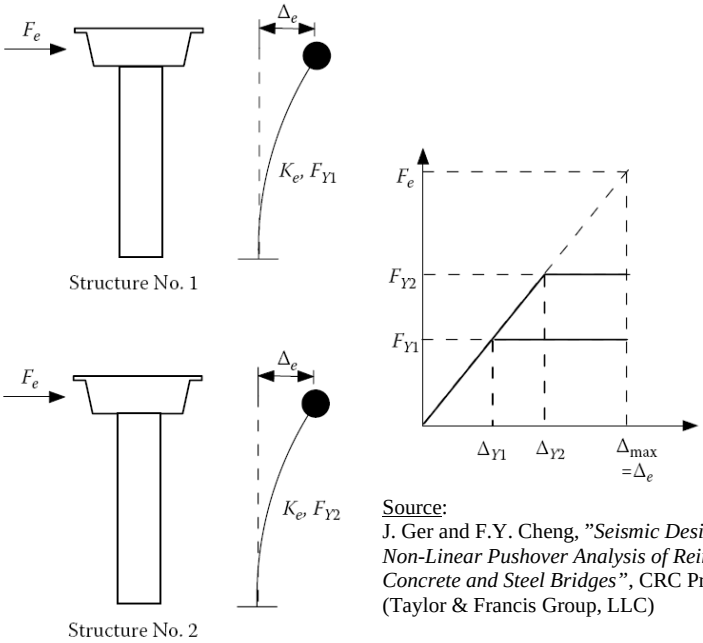
The R-factor method uses the equal displacement approximation which assumes that the maximum displacement of an elastic system is the same (or very close) to that of an inelastic system when subjected to the same design earthquake. For instance, based on the equal-displacement approximation the two structures shown in Figure 7.5.1-1, having the same lateral stiffness K_e, but with different yield strengths, F_{y1} and F_{y2}, subject to the elastic lateral force F_e, will have the same inelastic deformation δ_{max}.

In this case, the ductility demands of Structures 1 and 2 can be expressed as:

$$\mu_1 = \frac{\delta_{max}}{\delta_{Y1}} = \frac{F_e}{F_{Y1}} = R_1 \quad \text{, for Structure 1} \qquad (8.4.1\text{-}1)$$

$$\mu_2 = \frac{\delta_{max}}{\delta_{Y2}} = \frac{F_e}{F_{Y2}} = R_2 \quad \text{, for Structure 2} \qquad (8.4.1\text{-}2)$$

As seen from the above equations, the ratio of the elastic strength demand and the inelastic strength demand denotes the force-reduction factor, R. Moreover, considering the equal-displacement approximation assumption, the force reduction factors R_1 and R_2, likewise represents the member ductility demands, μ_1 and μ_2, respectively. 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.



Source:
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)

Figure 7.5.1-1 R-Factor Based on Equal Displacement Approximation

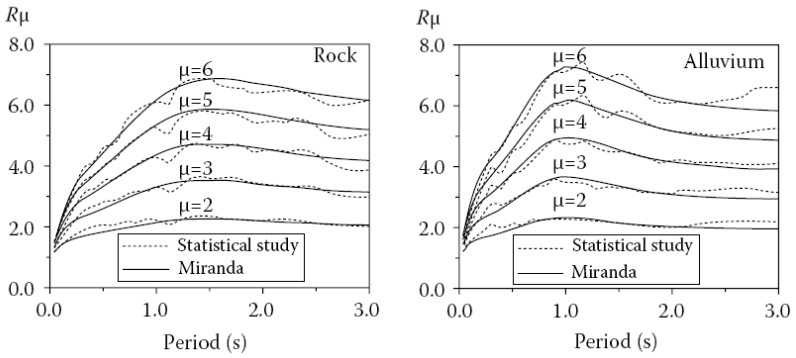
7.5.2 Drawback of the Force-Reduction R-Factor

The AASHTO traditional procedure to seismic design of bridges utilizes the force-based approach where structural damage is controlled by an assignment of a certain level of strength derived by application of the R-factor. However, the use of the R-factors generalizes the ductility capacity of members and tends to neglect the relationship between member strength and stiffness. This leads to the following drawbacks and issues in the use of the force-reduction factor[1]:

1) *J. Ger and F.Y. Cheng, "Seismic Design Aids for Non-Linear Pushover Analysis of Reinforced Concrete and Steel Bridges", CRC Press, 2012*

1. ***R-factor and structural period***. As indicated in Table 7.5.1-1, the R-factors are given for different substructure types, independent of the structural period. However, the R-factors are, in fact, a function of the structural period of vibration (T), the structural damping, the hysteretic behavior of the structure, the soil conditions at site, and the level of inelastic deformation (i.e. ductility demand). This can be seen in Figure 7.5.2-1 where the mean force-reduction factor spectrum for a single-degree-of-freedom system is plotted using recorded ground acceleration time histories on rock and on alluvium. As shown in the figure, the soil conditions at the site, particularly very soft soil, can have significant effect on the R-factor values. Further, the figure indicates that the ductility demand is larger than the force-reduction factor for short period structures which indicates that the equal-displacement approximation is not appropriate.

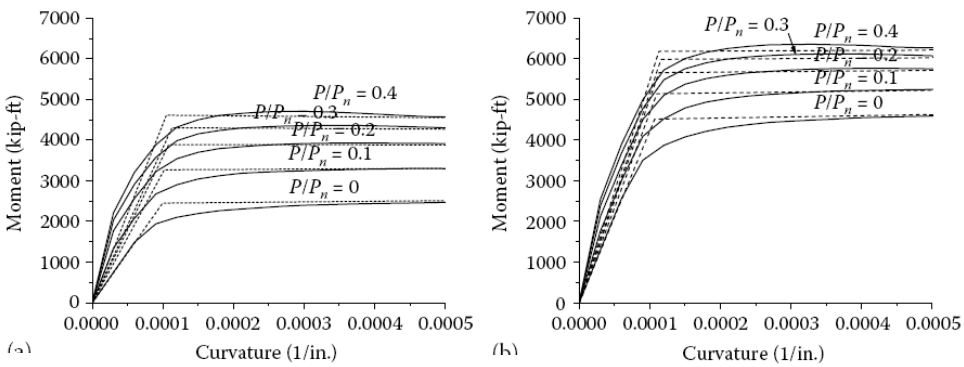
2. ***Member strength and stiffness***. As shown in Figure 7.5.1-1, ductility demand is obtained by the equal-displacement approximation, which assumes a constant member initial stiffness that is independent of the member's strength and that yield displacement or yield curvature is directly proportional to strength. On the contrary, the M-χ curves in Figure 7.5.2-2 indicate clearly that the initial stiffness of the member is not constant but a function of the moment capacity. The initial stiffness of the bilinear M-χ curves in the figure represents the cracked section flexural rigidity of the concrete member at which the first longitudinal steel reinforcement yield occurs and its intersection with the post-yield stiffness defines the location of the nominal moment M_n and the nominal curvature χ_n. Further, it can be seen that the stiffness is directly proportional to strength and that the yield displacement or curvature is independent of strength.



(a) Rock (b) Alluvium

Source: Miranda, E. and Bertero, V., Earthquake Spectra, 10(2), 357, 1994

Figure 7.5.2-1 Mean Force-Reduction Factors



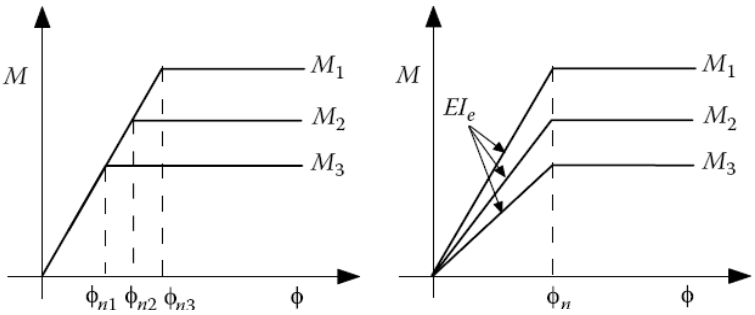
(a) Reinforcement Ratio = 1.4% (b) Reinforcement Ratio = 2.8%

Source: *J. Ger and F.Y. Cheng, "Seismic Design Aids for Non-Linear Pushover Analysis of Reinforced Concrete and Steel Bridges", CRC Press, 2012*

Figure 7.5.2-2 Moment-Curvature Curves of a 48" Circular Column

A comparison of the equal-displacement assumption with the realistic condition (Figure 7.5.2-3), assuming that yield displacement or curvature increases proportionally with strength, indicates the poor correlation between strength and ductility demand (as determined using the R-factor method). Moreover as seen in the figure, the nominal curvature, χ_n, is found to be independent of strength but rather dependent on the column size or diameter and the yield strain, ε_y, of the longitudinal reinforcement.

3. ***Elastic mode shapes to predict inelastic demand***. The current practice in force-based design is to apply the member stiffness at yield or the cracked section stiffness for ductile members in the elastic response spectrum analysis using the design acceleration spectrum provided in the code. Since this procedure does not consider the inelastic stiffness distribution of the member at the maximum inelastic response of ductile structures, the elastic mode shapes obtained in the analysis may be quite different with the actual inelastic mode shapes of the structure.



(a) Equal-Displacement (b) Realistic Model

Source: *J. Ger and F.Y. Cheng, "Seismic Design Aids for Non-Linear Pushover Analysis of Reinforced Concrete and Steel Bridges", CRC Press, 2012*

Figure 7.5.2-3 Moment-Curvature Relationship

4. ***Reliability in predicting bridge performance under strong ground motion***. Since the ductility demand of ductile members cannot be predicted accurately using the force-based approach, the performance level required for a bridge under the design earthquake may not be achieved. Moreover, since ductility is a poor indicator of damage potential, two bridge structures designed to the same specifications and with the same force-reduction factor or ductility factors may experience different levels of damage under a given earthquake.

7.6 Issues on Bridge Falling Down Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as "Bridge Falling Down Prevention System" based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently.

However, in NSCP and AASHTO, respective specifications for the functions to prevent bridge falling down such as minimum supporting length and restrainers are specified separately, but the efficient functionable system that various preventing mechanisms can complement and work together is not explicitly specified against large-scale seismic motion.
Therefore, in this article, the efficient system in JRA is introduced and proposed to be implemented into new seismic specifications.

7.6.1 Specified Devices/ Functions in NSCP

The provisions to prevent bridge falling down in NSCP basically follow the articles of seismic design requirements in AASHTO Standard Specifications for Highway Bridges, which is out-of-date philosophy in comparison with JRA and AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd EDITION 2011, hereinafter called the "AASHTO 2011". The descriptions of such the functions are limited only in the items of supporting length and horizontal linkages except related specification of vertical forces that may be caused in bearings.

(1) Supporting Length

To secure adequate supporting length at abutments and piers is the most important and well-understood provision to prevent bridge falling down. In NSCP, the article regarding the supporting length, the length of "N" in Figure 7.6.1-1 is specified

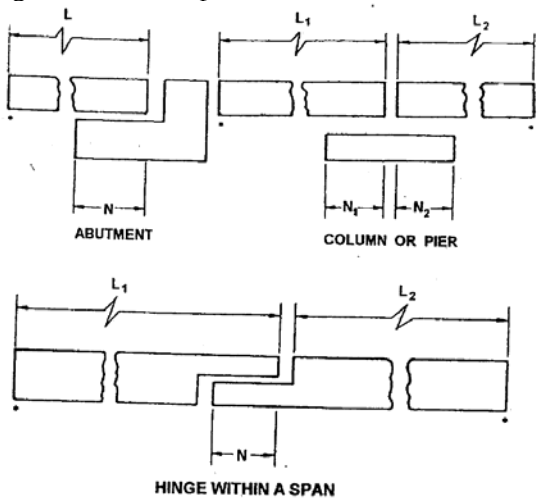


Figure 7.6.1-1 Dimension for Minimum Supporting Length in NSCP

The minimum supporting length is specified in NSCP by following the equation.

$$N = 305 + 2.5L + 10H$$

N: Minimum support length (m)

L: Length in meters of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck (m)

H: Average height of columns (m)

In case of general bridge with 40m of deck length and 5m of pier height, the minimum supporting length using NSCP code is approximately 45cm, which may be seriously insufficient length comparing to that of JRA, by which the minimum supporting length may be approximately 90cm.

(2) Longitudinal Restrainer

The provision to restrain relative displacements between the superstructure and the substructure is specified in NSCP using longitudinal restrainer that links between the superstructure and the substructure. However, the related article in NSCP specifies that the strainer must secure sufficient slack or expansion gap so that it must not start to act until the design displacement is exceeded, which may not be rational and precise philosophy on interlocking system with the supporting length. Thus, the function of the restrainer in NSCP may be specified individually and may not take into account the philosophy of minimum supporting length, which negates the efficient functional system of interlocking with other function sufficiently.

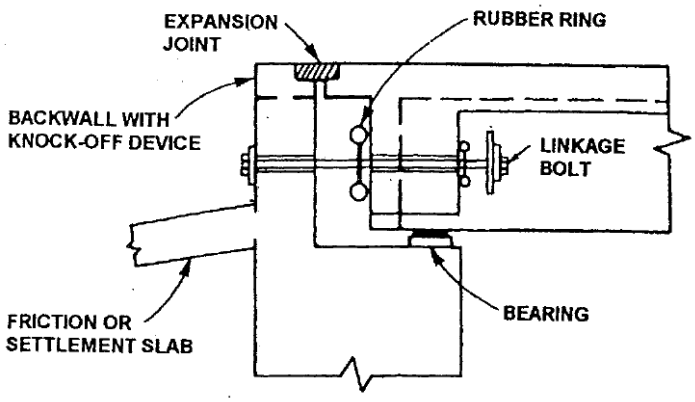


Figure 7.6.1-2 Longitudinal Restrainer in NSCP

As explained above, the specifications of the functions to prevent bridge falling down in NSCP basically follow the article of seismic design requirements in AASHTO Standard Specifications for Highway Bridges, which may be out-of-date philosophy in comparison with JRA and AASHTO 2011. Therefore, the design on new bridge and retrofitting/ strengthening will be carried out by adequately incorporating the latest design concept specified in AASHTO 2011 and FHWA Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges.

7.6.2 Specified Devices/ Functions in AASHTO

The provisions to prevent bridge falling down in AASHTO are basically specified in AASHTO 2011, in particular, for retrofitting/ strengthening of bridges, structural details complementing AASHTO 2011 are specified in FHWA Seismic Retrofitting Manual for Highway Structures Part 1 Bridges.

(1) Supporting Length

The article regarding supporting length, which may be one of the most important requirements which is divided into levels of seismic design categories, is also specified in AASHTO 2011.

The supporting length in seismic design category A, B and C is specified in AASHTO 2011 by following the equation.

<Seismic Design Category A, B, and C)

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2)$$

N: Minimum support length (in)
L: Length in ft. meters of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck
H: Average height in ft. of abutment and columns
S: Angle of skew of support measured from a line normal to span (degree)

In case of general bridge with 40m (131.2335ft) of deck length and 5m (16.4042ft) of pier height, the minimum supporting length is approximately 30cm (11.937in) using the specification, which may be seriously insufficient length comparing to that of JRA, by which the minimum supporting length is approximately 90cm.

Meanwhile, the supporting length in seismic design category D is given by following the equation.

<Seismic Design Category D)

$$N = (4 + 1.65\Delta_{EQ})\ (1 + 0.00025S^2) \geq 24$$

N: Minimum support length (in)

Δ_{EQ}: Seismic displacement demand of the long period frame on one side of the expansion joint (in)

S: Angle of skew of support measured from a line normal to span (degree)

The supporting length for the seismic design category D is calculated by seismic displacement demands in which the minimum supporting length is approximately 60cm (24in), which may be also insufficient length from the value specified in JRA.

(2) Longitudinal Restrainer

The provisions specified in AASHTO 2011 are similar to the function of unseating prevention device of Bridge Falling Down Prevention System specified in JRA; however, the design concept in AASHTO 2011 is different from the design concept of unseating prevention device in JRA in coordination with other function.

For critical or essential bridges in seismic design category C or D, AASHTO 2011 specifies that either one function of the longitudinal restrainer or extended supporting length should be considered, which does not specify the interaction of these functions. The extended supporting length specified is given by following the equation.

<Seismic Design Category D)

$$N = (4 + 2.0\Delta_{EQ})\ (1 + 0.00025S^2) \geq 24$$

Also, in case that the designer has selected to install the longitudinal restrainer not to secure the extended supporting length, the related article in AASHTO 2011 specifies that the strainer must secure sufficient slack or expansion gap so that it must not start to act until the design displacement is exceeded, which may not be rational to the supporting length.

Therefore, the fail safe function is not considered in contrast to Bridge Falling Down Prevention System specified in JRA that both elements such as the supporting length and unseating prevention device are functionally interacting rationally.

Additionally, the design concept of the restrainer based on large-scale earthquake is not specified taking into account the horizontal load capacity of piers and overload factors. Thus, this function may not respond adequately to unexpected states such as unexpected displacement by unpredicted seismic ground shake, destruction of surrounding grounds and complicated vibrations caused in structural members unexpectedly, etc.

7.6.3 Bridge Falling Down Prevention System in JRA

In JRA, for this functional system to prevent superstructure fall-down under large-scale earthquake, the detailed philosophy and articulate design concepts are explicitly specified in the “Bridge Falling Down Prevention System” which aims at multiple prevention mechanisms that can complement each other efficiently, based on accumulated data and experiences of large number of seismic damages.

Additionally, adequate fail-safe functions against unexpected states that could not be specified in the design code are considered in the Bridge Falling Down Prevention System in JRA, such as unexpected displacement by unpredicted seismic ground shaking, lateral spreading of surrounding grounds and complicated vibrations caused in structural members unexpectedly.

(1) Supporting Length

The supporting length in seismic in JRA is specified by following the equation.

$$S_{ER} = U_R + U_G$$
$$S_{EM} = 0.7 + 0.005\,l$$

S_{ER}: Required supporting length (m)
U_R: Maximum response displacement at bearing under level 2 seismic motion (m)
U_G: Relative displacement caused by ground strain (m)
S_{EM}: Minimum supporting length (m)
l : Span length (m)

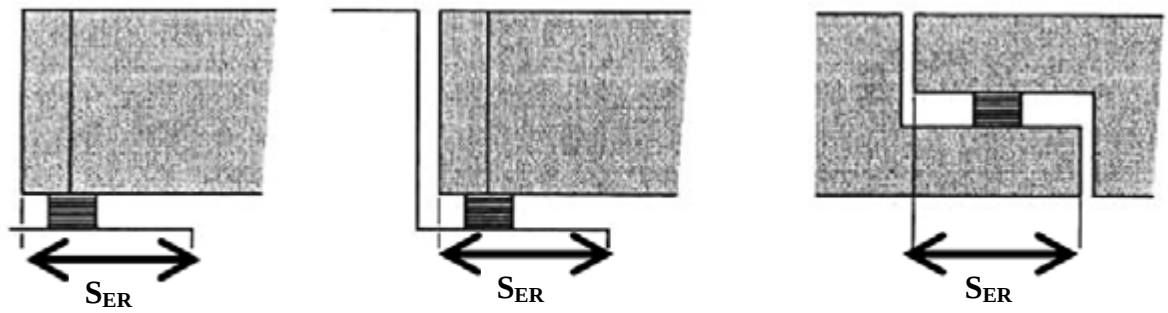


Figure 7.6.3-1 Supporting Length in JRA

The supporting length in JRA is obtained from the sum of relative displacement caused by ground strain and maximum response displacement at bearing under Level 2 seismic motion considering ground liquefaction adequately. The positive effects by other functions consisting of Bridge Falling Down Prevention System such as unseating prevention device and transversal displacement restrainer must not be considered in the determination of the supporting length from the viewpoint of failsafe function.

In the bridge design generally being implemented in Japan, the methodology to obtain structural behaviors such as horizontal response displacement of the deck by using nonlinear dynamic analysis is how common, so that the maximum response displacement should be calculated directly based on the dynamic analysis. However, if the required supporting length calculated by dynamic analysis remains small value in unexpectedly, the minimum supporting length is also specified.

As explained above, since the design concept of supporting length in JRA may be a rational function based on accumulated data and experiences from large number of seismic damages, we would like to propose to implement this methodology into new seismic specifications.

(2) Unseating Prevention device

The concept of this device in JRA is to prevent the relative displacement between the super and substructures from exceeding the supporting length, in case of failure or destruction of bearings under unexpected seismic forces. Several mechanisms are commonly utilized in Japan such as cable restrainer types, chain types, and stopper types.

Cable restrainer type Chain type

Figure 7.6.3-2 Examples of Unseating Prevention Devices in JRA

The expansion gap of the unseating prevention device is rationally specified in order to interlock with design concept of above-mentioned supporting length as 75% of the supporting length obtained from the sum of relative displacement caused by ground strain and maximum response displacement at bearing under Level 2 seismic motion considering ground liquefaction. Therefore, the unseating prevention device will function as an important role of fail-safe device complementing the supporting length.

Accordingly this device may secure fail-safe function that intends to become a final device against bridge falling down due to unexpected relative displacements. Thus, the general seismic design force, specified in NSCP, is adequate for the design of these important devices. Therefore, the design forces to be utilized for the unseating prevention devices in JRA, is not only the force utilized for the links between the superstructure and the substructure such as bearings, but the design force equivalent to the ultimate strength of the pier that the device is to be installed on should be specified because not only the device itself but also anchored members on the superstructure and the substructure must be capable of resisting sufficiently the secondary forces caused when the response of the superstructure is restrained in order that the unseating prevention device can function adequately.

Consequently, the unseating prevention device is not only the independently specified device in NSCP and AASHTO 2011 but may also be a functional device interacting with bearings and support length as a fail-safe function. This will be proposed to be included into the new seismic specifications.

(3) Transversal displacement restrainer

Assuming that the above mentioned unseating prevention device complements the supporting length for longitudinal direction as a fail-safe device, the transversal displacement restrainer is specified as the device to restrain abnormal displacements of the superstructure for transversal direction due to structural and geometrical response in case of failure of bearings under unexpected seismic forces. This device may be an independent device different from above mentioned two devices.

Figure 7.6.3-3 Example of Transversal Displacement Restrainer in JRA

Basically, since the supporting area for transversal direction on the pier is wider than that for longitudinal direction, the unseating prevention device may not be installed for transversal direction. However, in case that the superstructure can easily rotate without restraint from the abutment and adjacent girders with the possibility of the superstructure falling down due to the large response in the transversal direction, the transverse restrainer is specified generally from geometrical condition of the superstructure.

The design forces to be utilized for the transverse displacement restrainer in JRA, is not the force utilized for links between the superstructure and the substructure such as bearings but the design force equivalent to the ultimate strength for transversal direction of the pier that the device is to be installed on since anchored members on the superstructure and the substructure must be capable to resist sufficiently the secondary forces resulting from the response of the superstructure.

Thus, this device has a characteristic function that prevents bridge falling down in the transversal direction taking account the geometrical condition of the superstructure, which is not specified explicitly in NSCP and AASHTO. This system is thus proposed to be included into the new seismic specifications.

CHAPTER 8 APPROACH TO THE DEVELOPMENT OF LOCALIZED SEISMIC ACCELERATION RESPONSE SPECTRA FOR BRIDGE DESIGN

8.1 Method 1 – Based on AASHTO Acceleration Response Spectra (Currently Utilized by DPWH)

8.1.1 Purpose of the Development

(1) Background of the Development

- Seismic loads, inertial forces, utilized in seismic design are definitely described by Seismic Acceleration Response Spectra.
- In JRA, Response Spectra of each soil type are determined in consideration of spectral analysis on large number of acceleration waveforms, obtained from measurement data of large-scale earthquake, on the ground surface.
- In Philippines, strong-motion waveforms required for spectral analysis commonly executed in Japan are not recorded.
- The seismic acceleration response spectra in AASHTO is standardized based on the maximum acceleration (A) of ground surface; the maximum acceleration of the ground surface in NSCP is specified as 400 gal.

(2) Purpose of the Development

The objectives of the development of the acceleration response spectra are to:

- Confirm whether the acceleration spectra of each soil type, specified in AASHTO, can be adopted into the ground properties of Philippines.
- Study design earthquake motions reflecting Philippine conditions (local conditions).
- Propose standard acceleration spectra fit for the Philippines.

8.1.2 Development Procedure/Flowchart

(1) Flowchart

The development flowchart is shown in Figure 8.1.2-1 below:

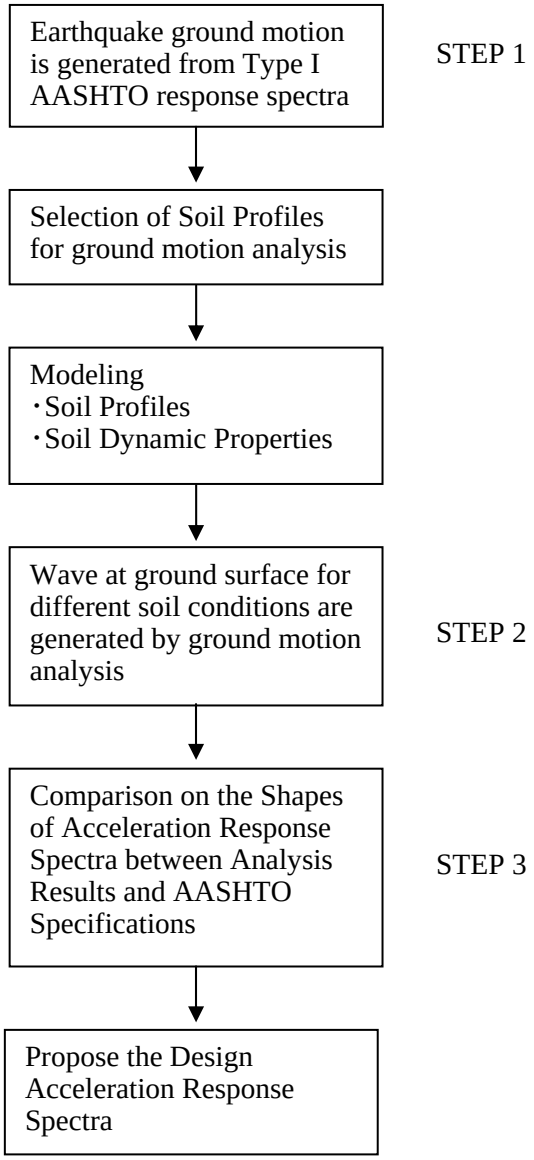


Figure 8.1.2-1 Acceleration Response Spectra Development Flowchart

(2) Procedure (STEP1)

1) Acceleration response spectra of Soil Type I of AASHTO can be specified based on the earthquake motions observed on fully hard ground or rock (outcrop)
2) From this spectrum, incident wave can be created.

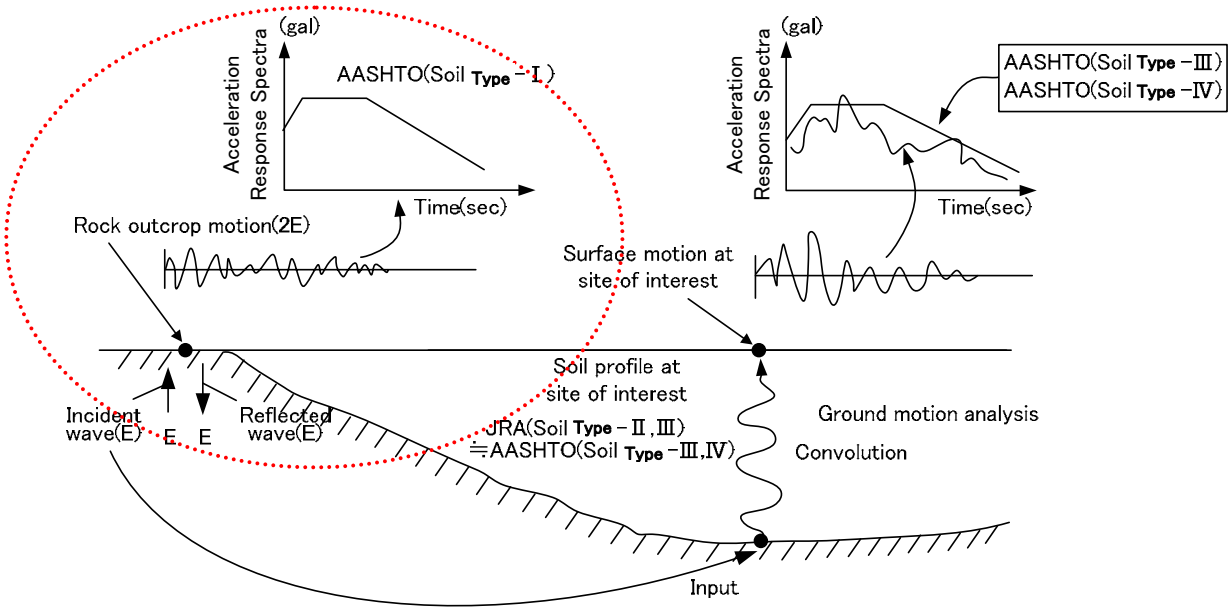


Figure 8.1.2-2 Procedure (STEP1)

(3) Procedure (STEP2)

Using this incident wave acceleration, wave at the ground surface of various soil conditions (Soil Type II: JRA, Soil Type III : JRA) are to be calculated by means of ground motion analysis.

※ Soil Type II (JRA) ≒ Soil Type III (AASHTO)

※ Soil Type III (JRA) ≒ Soil Type IV (AASHTO)

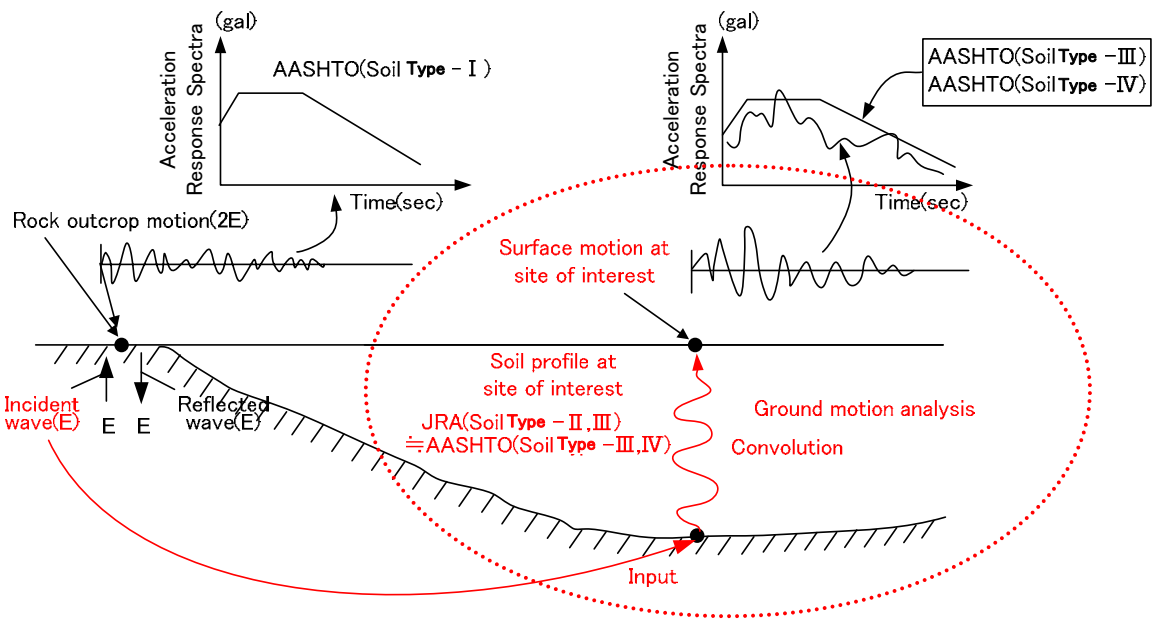


Figure 8.1.2-3 Procedure (STEP2)

(4) Procedure (STEP3)

Using the above waves at ground surface, acceleration response spectra by the ground types can be created for comparative study with standard spectra specified in AASHTO.

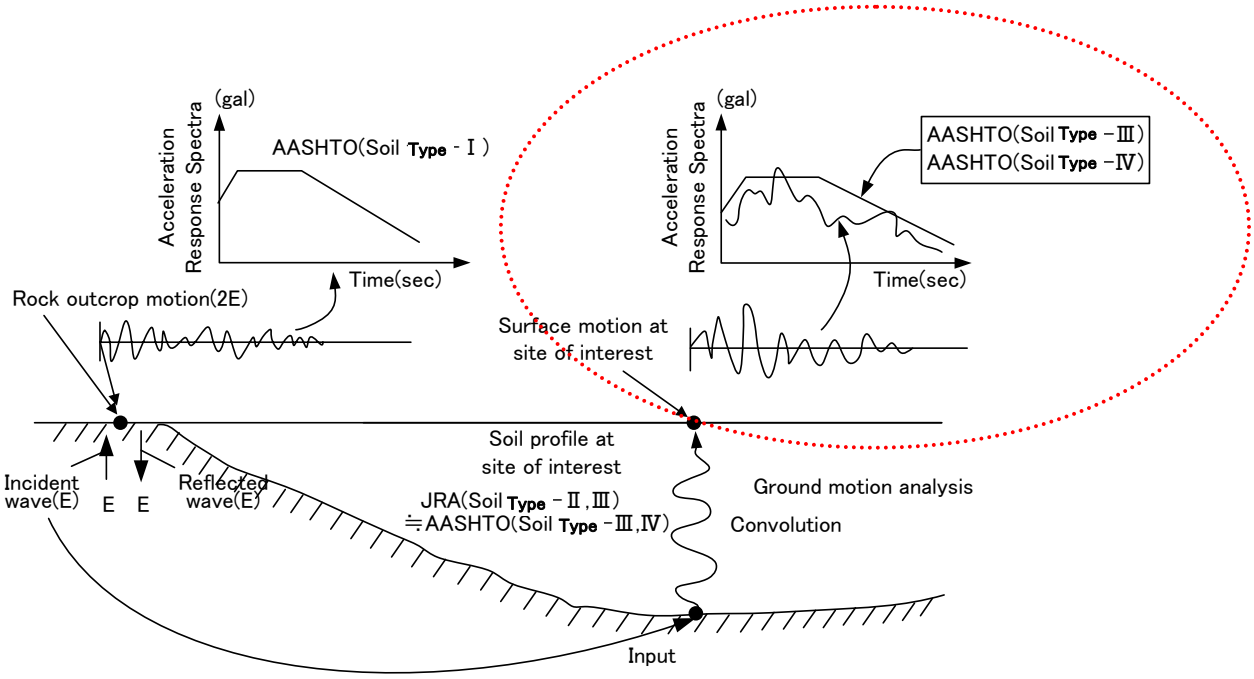


Figure 8.1.2-4 Procedure (STEP3)

8.1.3 Conversion from Acceleration Response Spectra to Earthquake Ground Motions

(1) Scaling and Spectrum Matching

Figure 8.1.3-1 illustrates the flowchart showing the procedure for developing earthquake ground motion matching the target spectrum. As shown in the flowchart, earthquake ground motion is developed by adjusting the amplitude of observed wave data iteratively so that the acceleration response spectrum of the observed earthquake ground motion matches the target acceleration response spectrum.

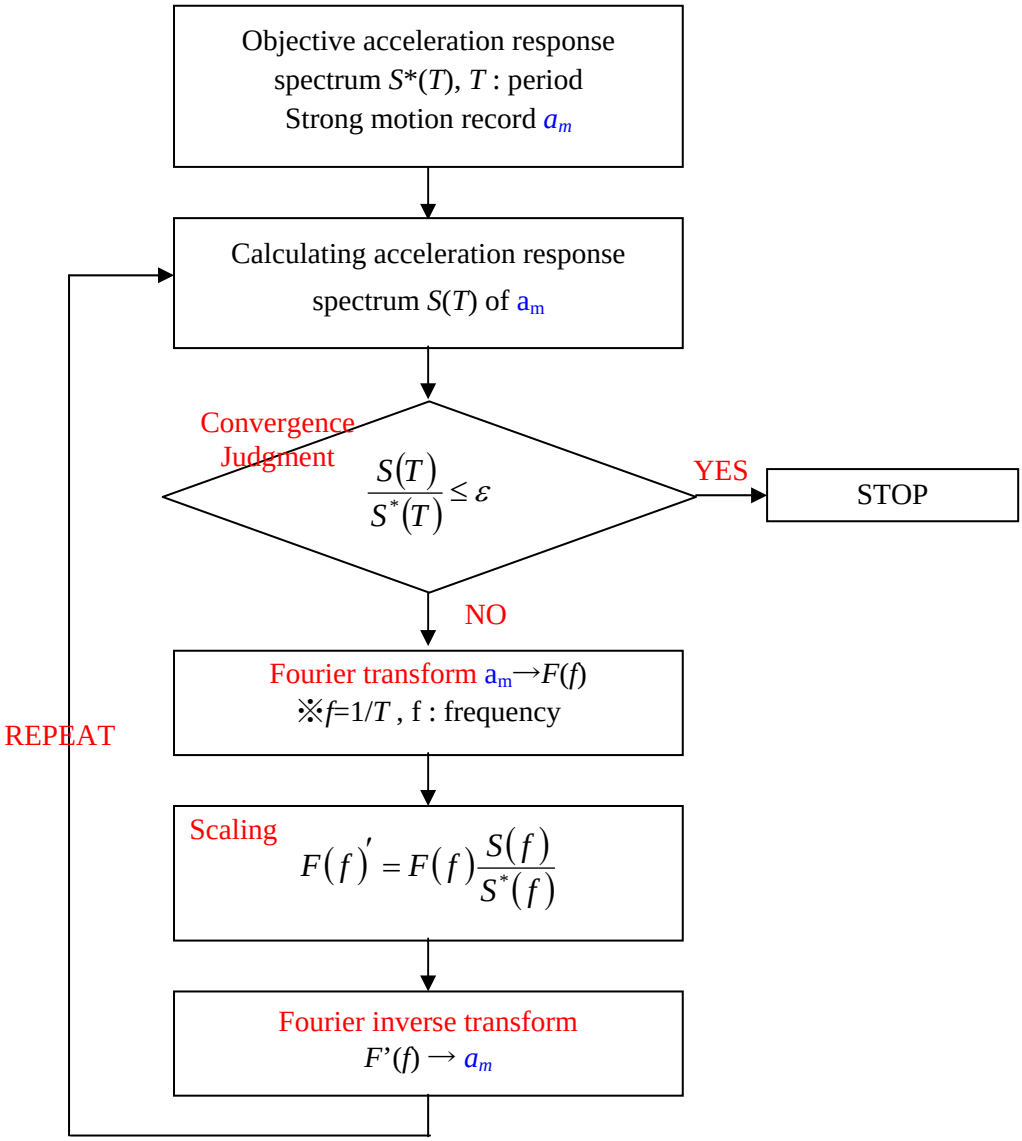


Figure 8.1.3-1 Flowchart for Developing Earthquake Ground Motion Matching the Target Spectrum

(2) Target Spectra (AASHTO Soil Type-1)

The following figure shows the target acceleration response spectrum specified in AASHTO. The spectrum indicates around 1000gal up to 0.3sec and decreases with increment of natural period from 0.3sec.

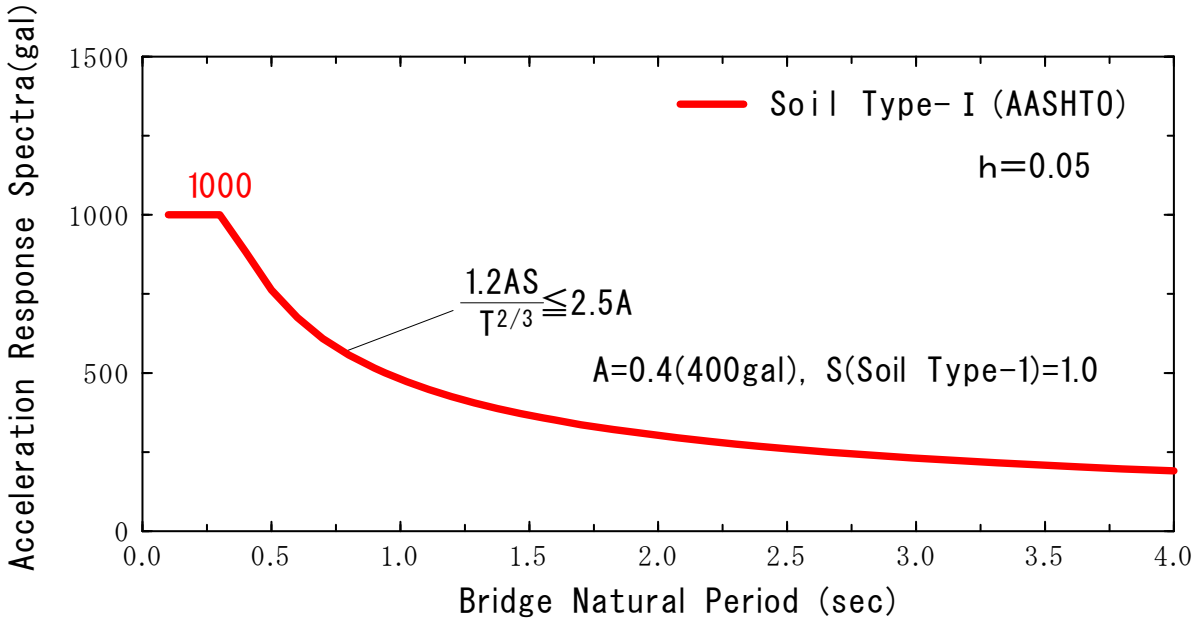
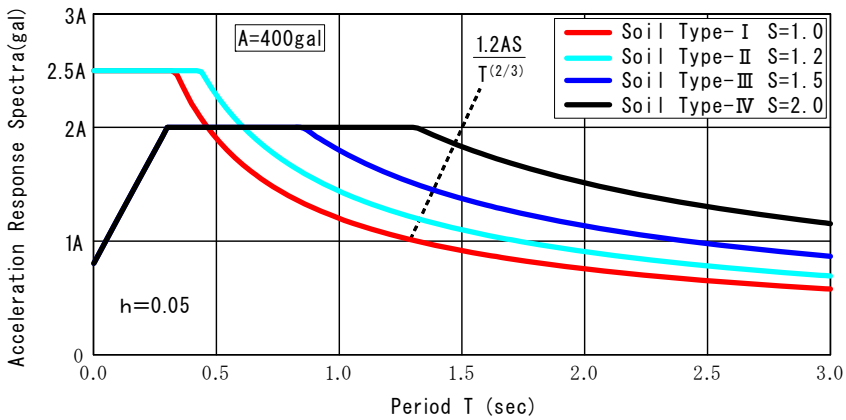


Figure 8.1.3-2 Target Spectra (AASHOTO 2007, Soil Type-Ⅰ)



For Reference: Design Spectra (AASHTO 2007)

Figure 8.1.3-3 Design Spectra (AASHTO 2007)

Table 8.1.3-1 Definition of Soil Profile Types (AASHTO 2007)

Soil Profile Types	Description of Soils
Soil Profile Type I (Referred to as Soil Type-I)	A profile shall be as Type I if composed of: ・Rock of any description, either shale-like or crystalline in nature, or ・Stiff soils where the soil depth is less than 60000mm, and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.
Soil Profile Type II (Referred to as Soil Type-II)	A profile with Stiff cohesive or deep cohesionless soils where the soil depth exceeds 60000mm and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.
Soil Profile Type III (Referred to as Soil Type-III)	A profile with soft to medium-stiff clays and sands, characterized by 9000mm or more of soft to medium-stiff clays with or without intervening layers of sand or other cohensionless soils shall be taken as Type III.
Soil Profile Type IV (Referred to as Soil Type-IV)	A profile with soft clays or silts greater than 12000mm in depth shall be taken as Type IV.

Table 8.1.3-2 AASHTO soil Types and Corresponding JRA Soil Types

AASHTO 2007	JRA
Soil Type-I	Soil Type-I
Soil Type-II	
Soil Type-III	Soil Type-II
Soil Type-IV	Soil Type-III

(3) Seed Earthquake Records Selected as Rock Outcrop Motion

For the strong motion records to be adapted into the target spectra, the records observed in Japan are recommended to be utilized since the number and types of earthquake have been recorded abundantly in Japan. Philippines and Japan are both surrounded by plates, so that the earthquake characteristic due to inland active faults and the mechanism and environment are very much similar. The intended strong motions are the records observed in large-scale earthquakes which cause huge damages in Japan in these years.

Table 8.1.3-3 Strong Motion Seismograph Networks (Database)

No.	Eq. Name	Database
EQ1	Tottori-ken Seibu EQ.	K-Net (JAPAN)
EQ2	Geiyo EQ.	K-Net (JAPAN)
EQ3	Miyagi-ken Oki EQ.	K-Net (JAPAN)
:		K-Net (JAPAN)
EQ17	Naganoken Chuubu EQ.	K-Net(JAPAN)

Table 8.1.3-4 Seed Earthquake Records Selected as Rock Outcrop Motion

No.	Date	EQ. Name	Tectonics	Style	Magnitude
EQ 1	Oct 6, 2000	Tottori-ken Seibu EQ.	Crustal	SS	Mw6.7
EQ 2	Mar 24,2001	Geiyo EQ.	Subduction	N	Mw6.8
EQ 3	May 26,2003	Miyagi-ken Oki EQ.	Subduction	N	Mw7.0
EQ 4	Jul 26,2003	Miyagi-ken Hokubu EQ.	Crustal	R	Mw6.0
EQ 5	Sep 26,2003	Tokachi Oki EQ.	Interplate	R	Mw8.3
EQ 6	Oct 23,2004	Niigata-ken Chuuets EQ.	Crustal	R	Mw6.6
EQ 7	Mar 20,2005	Fukuoka-ken Seiho Oki EQ.	Crustal	SS	Mw6.6
EQ 8	Aug 16,2005	Miyagi-ken Oki EQ.	Interplate	R	Mw7.2
EQ 9	Mar 25,2007	Noto-kanto EQ.	Crustal	R	Mw6.7
EQ10	Jul 16,2007	Niigata-ken Chuuets Oki EQ.	Crustal	R	Mw6.6
EQ11	Jun 14,2008	Iwate-Miyagi Nairiku EQ.	Crustal	R	M_W6.9
EQ12	Jul 24,2008	Iwate-ken Engan Hokubu EQ.	Subduction	N	M_W6.8
EQ13	Aug 11,2009	Suruga-wan EQ.	Subduction	R	M_W6.2
EQ14	Mar 11,2011	Touhoku-chiho Taiheiyo Oki EQ.	Interplate	R	M_W9.1
EQ15	Mar 12,2011	Nagano-Niigata Kenzakai EQ.	Crustal	R	M_W6.3
EQ16	Apr 11,2011	Fukushima-ken Hamadori EQ.	Crustal	N	M_W6.6
EQ17	Jun 30,2011	Nagano-ken Chuubu EQ.	Crustal	SS	M_W5.0

NOTE:
SS : Strike Slip, R : Reverse (Trust) Fault, N : Normal Fault
Crustal (Crustal Earthquake): Crustal earthquake occur along faults, or breaks in the earth's crust
Subduction (Earthquakes within subducting plates): Extensive collapse can occur within subducting plates from sea trenches or elsewhere, and this sometimes causes large earthquakes.
Interplate (Interplate earthquakes): Interplate earthquakes occur along the interface between tectonic plates.

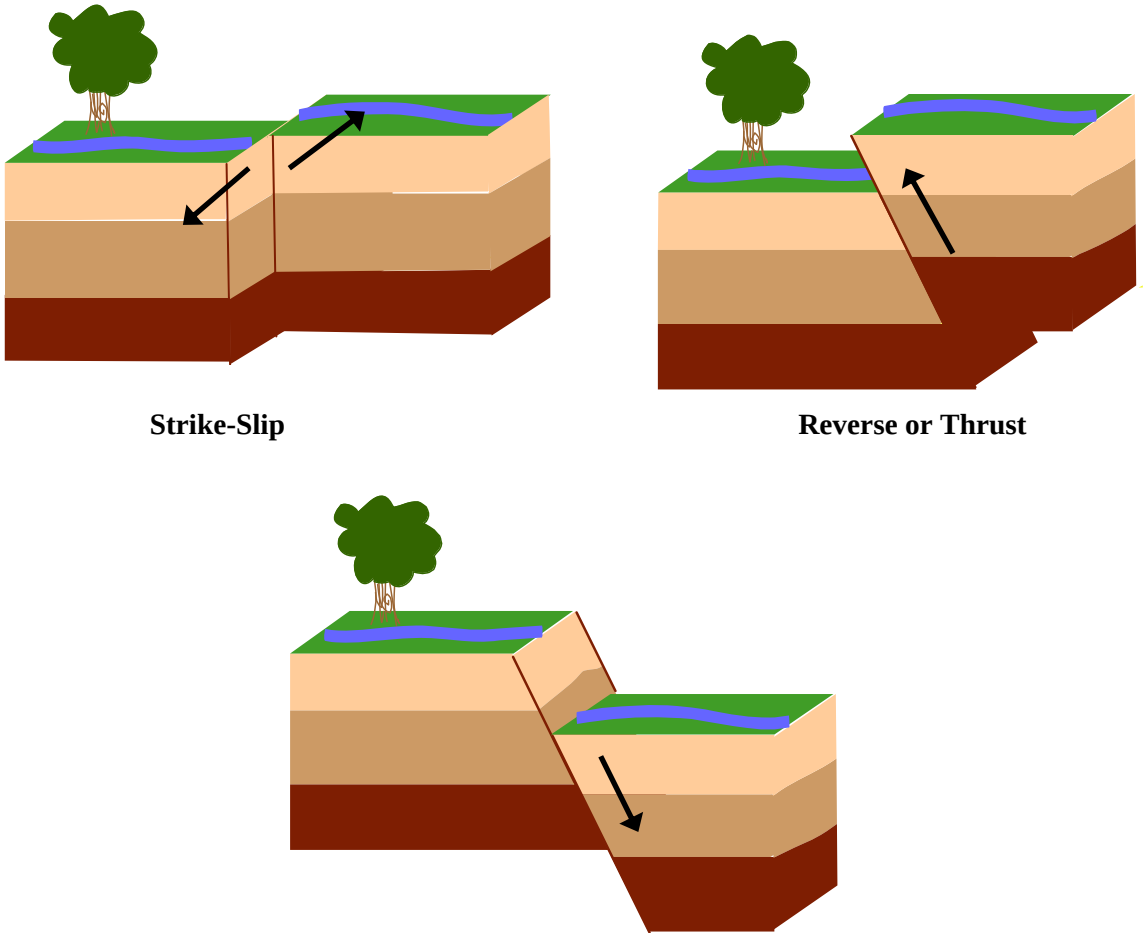


Figure 8.1.3-4 Three Types of Faults

8.1.4 Objective Soil Layer Conditions

(1) Objective Soil Layer Conditions

- As shown in Figure 8.1.4-1, ground analyses are conducted at a total of twelve places, such as six locations each for Types II and III in JRA (Types III and IV in AASHTO).
- Attention is given, therefore, to a number of different types of ground with different natural periods as much as possible.
- The ground of interest is located along the Pasig River running through the heart of Metro Manila and the Marikina River, a tributary of the Pasig River.
- In the area along the Pasig and Marikina rivers, boring surveys have been conducted extensively for river channel improvement projects. As a result, an abundance of boring survey results is available, and there is a vast accumulation of information on the N-value (SPT blow count) and geological formations at many sites, which is necessary for ground analysis[1].
- In selecting ground types, priority was given, wherever possible, to the ground near bridges on the Pasig and Marikina rivers, which is included in the scope of the soundness or earthquake resistance studies for Package B of this project, or the ground in adjacent areas.

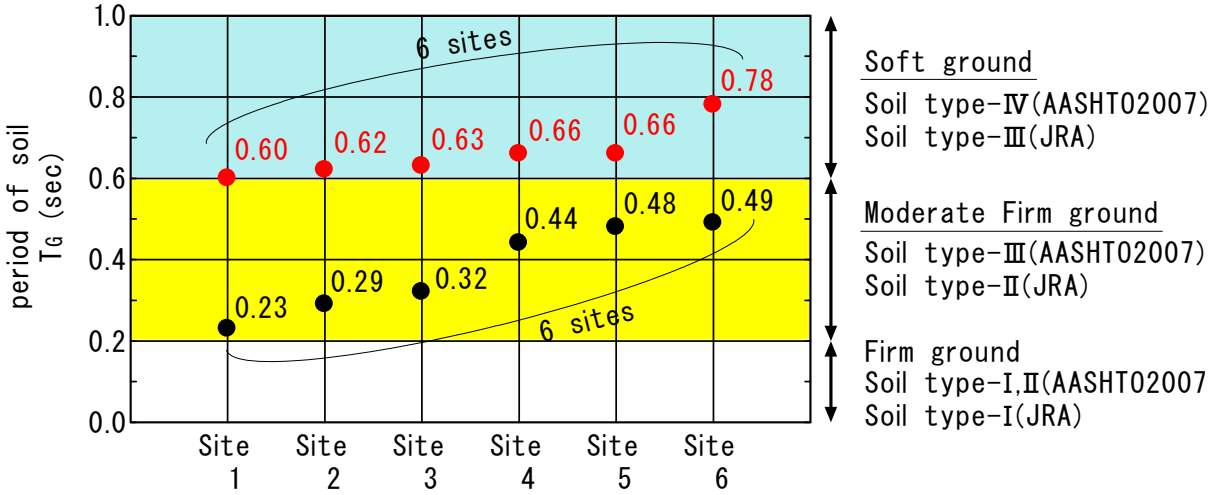


Figure 8.1.4-1 Natural Periods of Ground of Interest

[1] DETAILED ENGINEERING DESIGN OF PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT, GEOLOGY AND SOIL MECHNICS, VOLUME V, March 2002.
The Consulting Service for DETAILED ENGINEERING DESIGN OF PASIG-MARIKINA RIVER CHANNEL INPROVEMENT PROJECT was been carried out by CTI Engineering co., LTD. In association with NIKKEN Consultant Inc., WOODFIELDS inc. and Basic Technology and Management Corp. in close coordination with Department of Public Works and Highways from October 2000 to March 2002 under Loan No. PH-210 agreed between the Government of Republic of Philippines and Government of Japan through the Japan Bank for International Corporation.

Table 8.1.4-1 Types and Locations of Ground (Soft Ground) of Interest

Site No.	Period of Soil T_g(sec)	Soil Type (JRA)	Soil Type (AASHTO)	On / or near Bridge
1	0.60	III	IV	B05 Ayala Br.
2	0.62	III	IV	B07 Pandacan Br ～ B08 Lambingan Br.
3	0.63	III	IV	B06 Nagtahan Br.
4	0.66	III	IV	B02 Jones Br.
5	0.66	III	IV	B04 Quezon Br.
6	0.78	III	IV	B05 Ayala Br.

Table 8.1.4-2 Types and Locations of Ground (Moderate Firm Ground) of Interest

Site No.	Period of Soil T_g(sec)	Soil Type (JRA)	Soil Type (AASHTO)	On / or near Bridge
1	0.23	II	III	B14 Rosario Br.
2	0.29	II	III	B10 Guadalupe Br.
3	0.32	II	III	B09 Makati Mandaluyong Br.
4	0.44	II	III	B11 C-5 Br.
5	0.48	II	III	B08 Lambingan Br.
6	0.49	II	III	B06 Nagtahan Br.

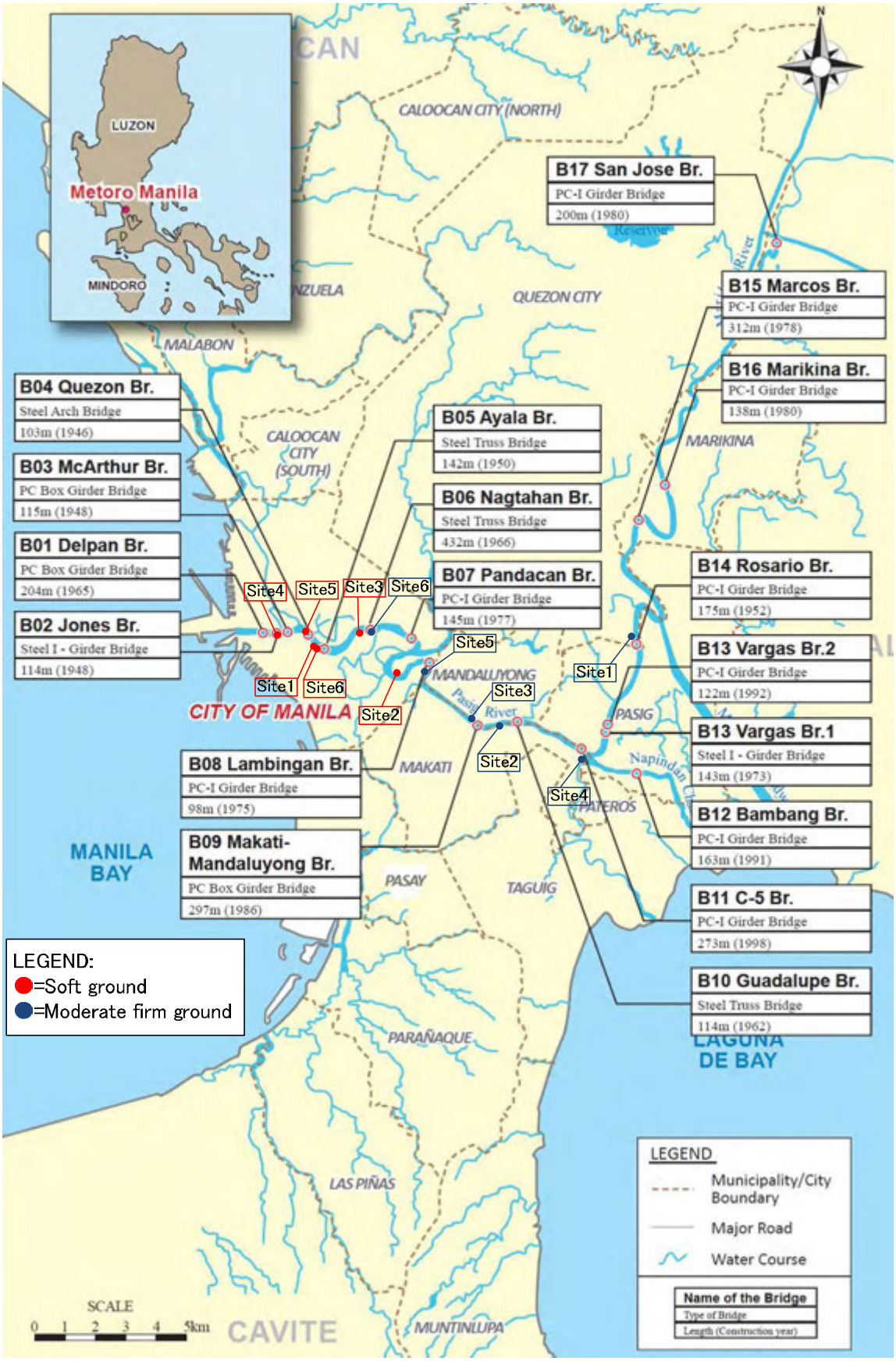


Figure 8.1.4-2 Locations of Ground of Interest

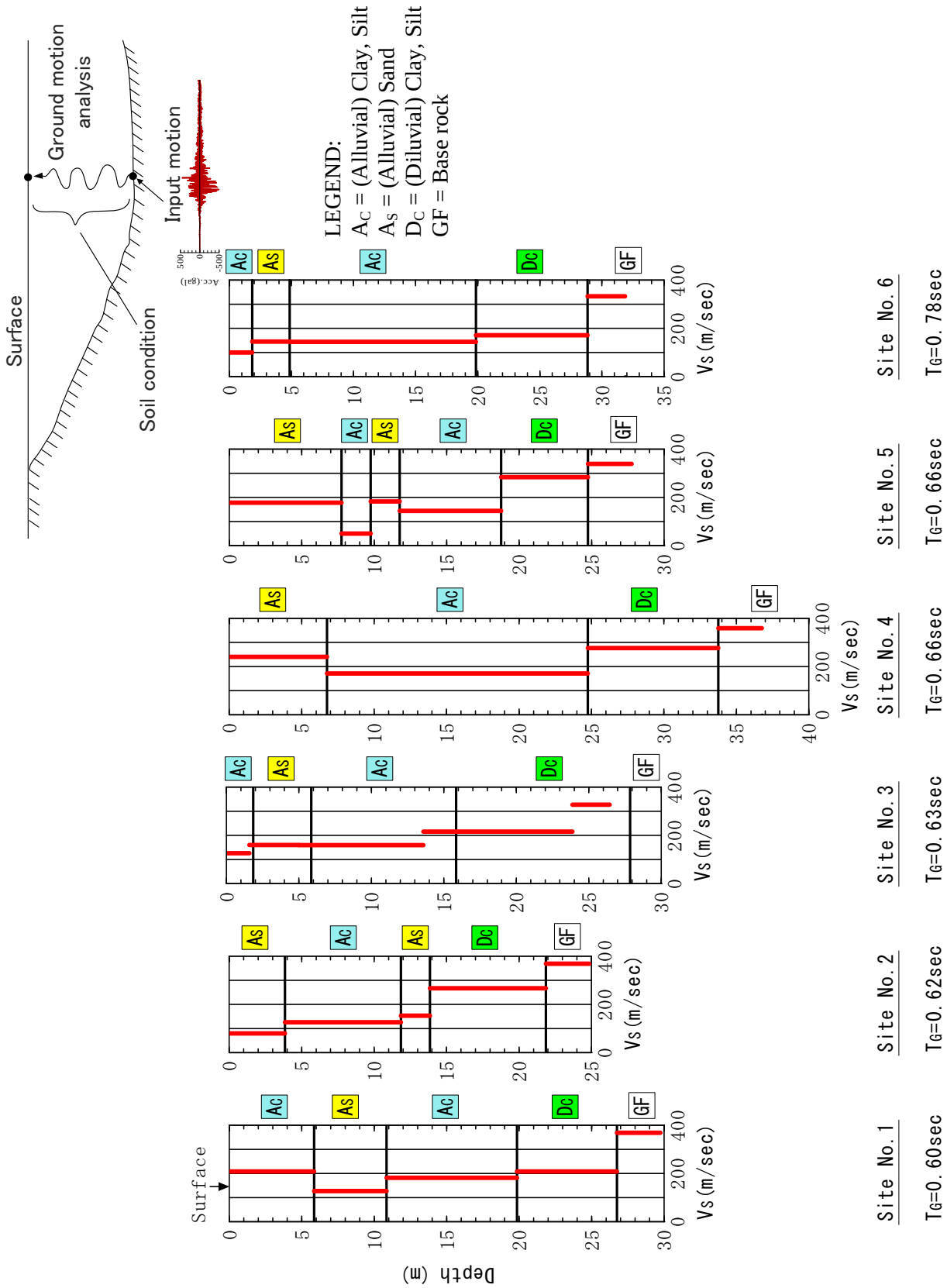


Figure 8.1.4-3 Soil Layer Conditions of Site (Soft Ground)

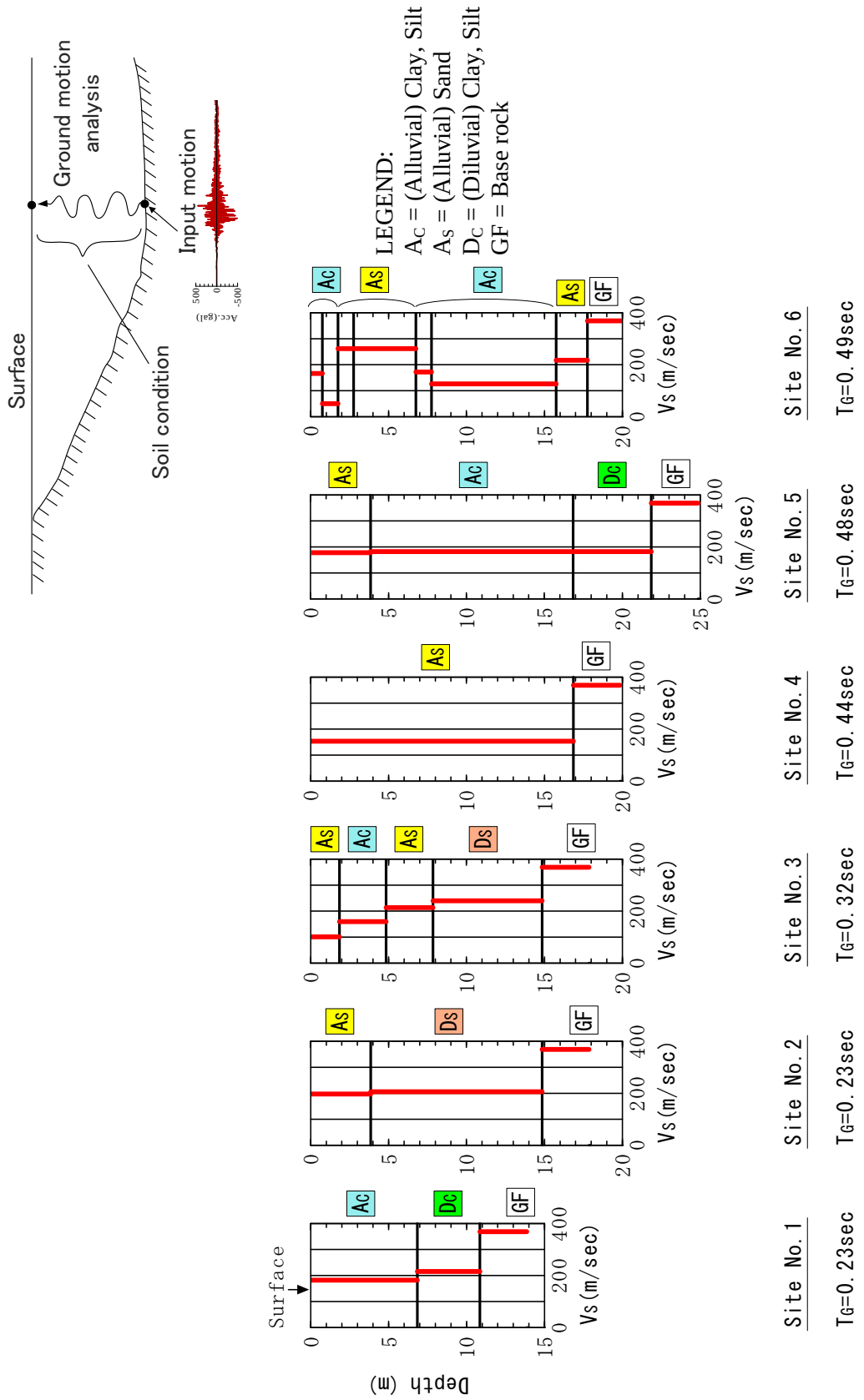


Figure 8.1.4-4 Soil Layer Conditions of Site (Moderate Firm Ground)

(2) Information about the Ground of Sites

- Bore hole data each site exist.
  1) N-value, 2) Soil classification (Clay, Sand)

- PS logging data (Shear velocity Vs) each site does not exist.

※PS Logging is already recommended on 2nd screening.

- Data on Non-linear stress-strain characteristics (G/G0-γ, h-γ) of ground in Metro Manila are unavailable.

- Shortfall data are covered by using proposed formulas obtained by previous studies. The proposed formulas utilized in Japan are to be applied, in which the formulas have covered abundant ground properties from firm and soft grounds.

1) Evaluation of Shear Velocity Vs

Vs (elastic wave velocity) can be estimated from Eq. (8.1.4-1) using the N value. Eq.(1) is an estimation equation derived from experimental result (JRA).

For cohesive soil layer

$$Vs = 100\ N^{1/3} \quad (1 \leqq N \leqq 25)$$

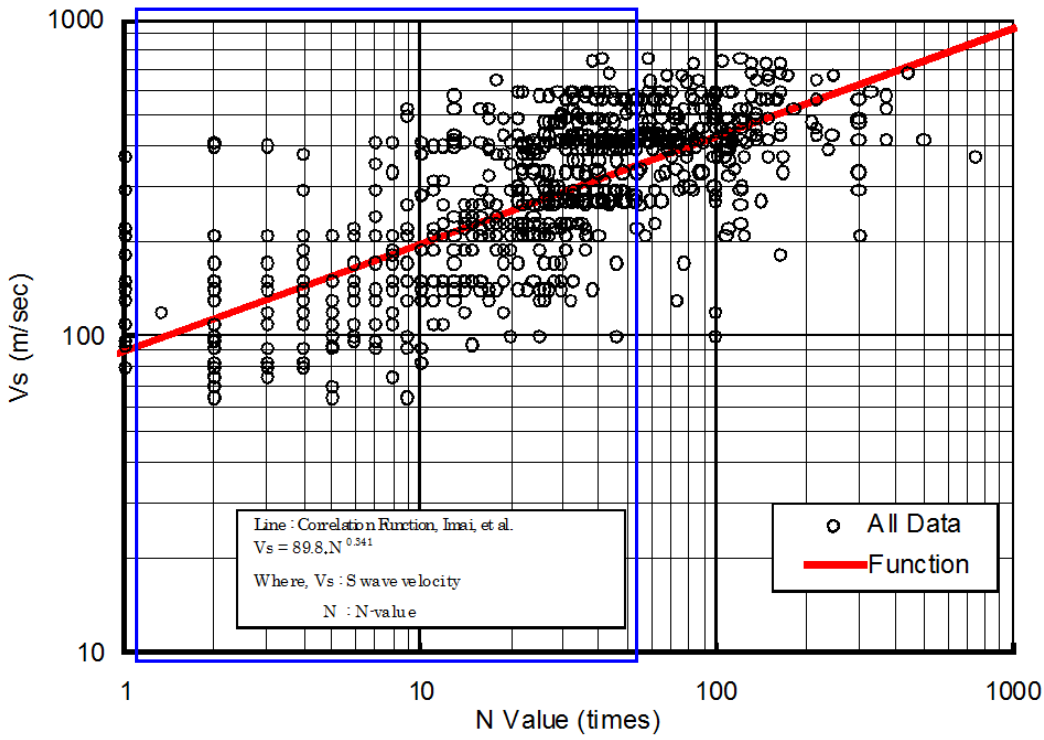
For sandy soil layer (8.1.4-1)

$$Vs = 80\ N^{1/3} \quad (1 \leqq N \leqq 50)$$

When the N value is =0, Vs=50m/s can be taken.

<u>For reference) Investigation Results of other Project in Philippines</u>.

Just for reference, an example of the relation between the N-value and V_S used for application to the ground in the Philippines (Eq. (8.1.4-1) is derived for application to the ground in Japan), is shown in Figure 8.1.4-5. The correlation widely applied in Japan is also shown in the figure. As indicated, these show similar relations. It is to be noted here that N-value–V_S relations proposed in Japan such as Eq. (8.1.4-1) are evaluation formulas proposed for engineering applications such as civil engineering and building construction. Relationships for data involving N-values of 50 or more as shown in Figure 8.1.4-5, therefore, have little significance from the engineering's point of view.



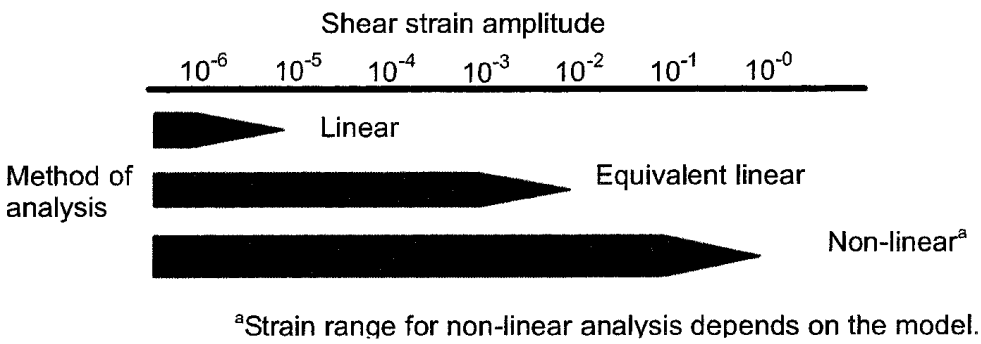
Note: The area surrounded by a blue line shows N Value≦50.
Source: Earthquake Impact Reduction Study for Metropolitan Manila in the Republic of the Philippines. March 2004, JICA, MMDA, PHIVOLCS

Figure 8.1.4-5 Relationship between N-Value and Shear Wave Velocity

8.1.5 Dynamic Analysis Methodology for Surface Soil Layers

(1) Method of Dynamic Analysis

- A constitutive model should be selected on the basis of expected strain level in the soil deposit. The limit in applicability of the equivalent linear model is 1%-2% strain at the maximum; nonlinear hysteresis model is appropriate over the largest strain range.
- Nonlinear Hysteresis Model is utilized to evaluate large strains on the ground in large-scale earthquake.



(2) Nonlinear Response Analysis

- Sites are assuming to be horizontally layered, and the soil profiles are modeled as a series of lumped masses connected by shear springs and dashpots. A simplified mechanical model of the soil deposit is shown in Figure 8.1.5-2.

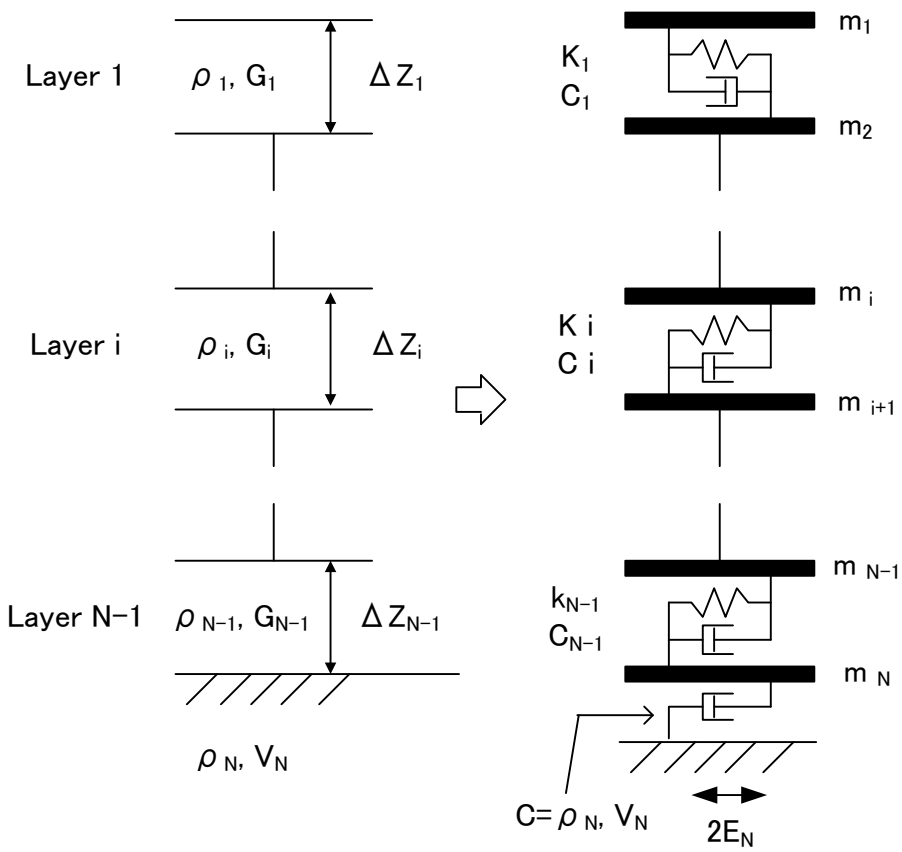


Figure 8.1.5-2 Non-Linear One-Dimensional Dynamic Analysis

- The lumped masses, m_i , are computed using the following equation assuming that the soil profile has a unit thickness perpendicular to the paper:

$$m_1 = \frac{\rho_1 Z_1}{2}\ ,\ \ m_N = \frac{\rho_{N-1} Z_{N-1}}{2} \text{ and } m_i = \frac{\left(\rho_{i-1} Z_{i-1} + \rho_i Z_i\right)}{2},\ \ i=2,3,\ldots,N\text{-}1$$

  Where m_i is the lumped mass assigned for the i-th layer having a density ρ_i with a thickness Z_i.

- The lumped masses are interconnected by springs and viscous damper elements which model the stiffness and damping of the soil deposit during horizontal displacement. The spring stiffness k_i of layer i can be obtained by considering the shear deformation of soil layer. With a soil column of unit section area, a height of h_i and G as the shear modulus, the spring stiffness, non-linear, is equal to

$$k_i = \frac{G_i}{Z_i} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (8.1.5\text{-}1)$$

- The nonlinear time domain response analysis is carried on by step-by-step numerical integration using the Newmark's β-method.

- The equation of motion for all N-1 soil layers in matrix form is given by:

$$M\ddot{u} + C\dot{u} + Ku = -M\ddot{u}_g \quad \cdots\cdots (8.1.5\text{-}2)$$

in which M, C, and K are, respectively, the mass, damping, and stiffness matrices for the soil deposit; $\ddot{u}_g$: acceleration at the base rock level, u: relative displacement on the base. To obtain the total (absolute) free field acceleration, the base rock acceleration is added to the acceleration of the top node.

- For strong ground motion, the non-linear behavior of the soil should be taken into account in the ground response analysis. For non-linear response analysis, the equation of motion (Equation 8.1.4-2) is modified as:

$$M\ddot{u} + C\dot{u} + R(u) = -M\ddot{u}_g \quad \cdots\cdots (8.1.5\text{-}3)$$

in which $R(u)$ is non-linear force-displacement relationship of the soil under cyclic loading. This non-linear relationship is represented by a skeleton curve as explained in Section 8.1.6.

- The damping matrix is given by:

$$C = \beta K$$

$$\beta = \frac{h}{\pi f}$$

in which h: damping ratio, h is assumed to represent damping in soil at initial conditions ($\gamma=10^{-6}$). $f=1/T$, T: fundamental ground period.

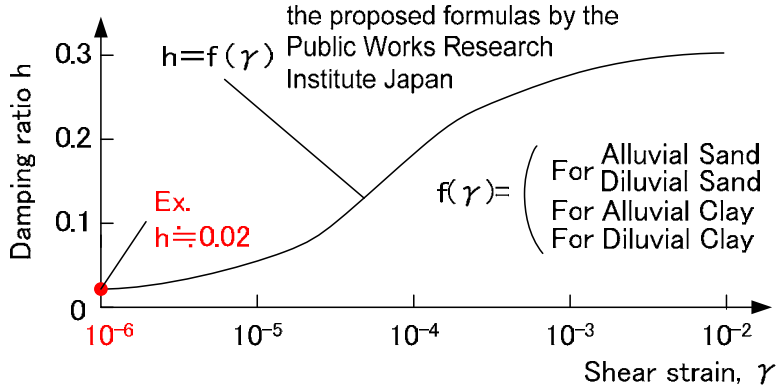


Figure 8.1.5-3 Damping in Soil at Initial Conditions ($\gamma=10^{-6}$)

(3) Input Motion

The input motion is assumed to be acting at an exposed surface of base layer by considering a transmitting boundary represented by a fictitious dashpot.

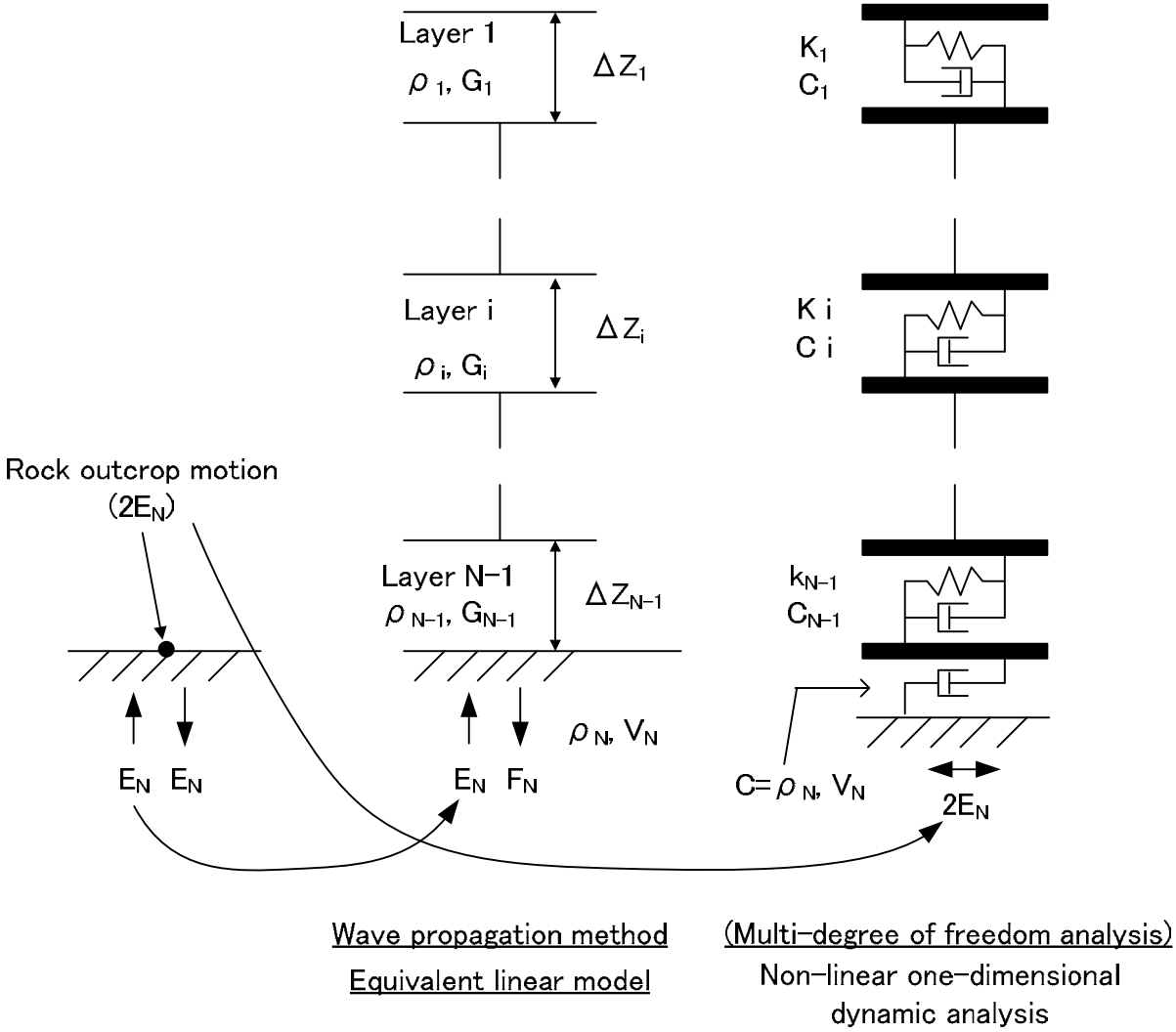


Figure 8.1.5-4 Wave Propagation Method and Multi-Degree of Freedom Analysis

8.1.6 Modeling of Soil Dynamic Properties

(1) H-D Model

- As nonlinear model of grounds, H-D model is utilized, which is frequently applied in a large number of fields of geotechnique.
- H-D model was proposed by Hardin and Drnevich as a method to organize the experimental results of wide variety materials from clay to sand.
- Stress-strain models for analysis are usually composed of two types of curves. A skeleton curve is one at virgin loading, whereas a hysteresis curve is one after unloading or reloading. A hysteresis curve is usually derived from a skelton curve by applying Masing's rule.

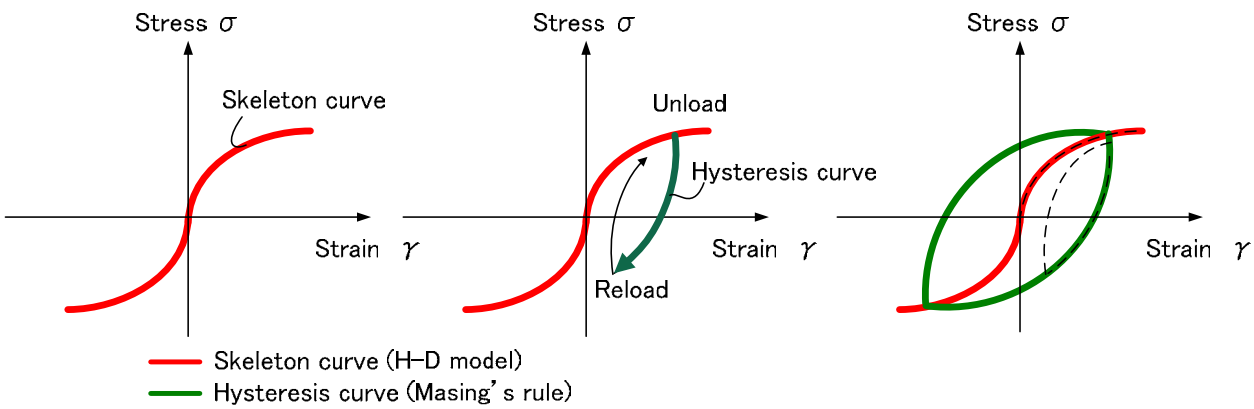


Figure 8.1.6-1 H-D model (Hardin and Drnevich)

(2) Nonlinear Model Parameters

Figure 8.1.6-2 shows the nonlinearity of the H–D model. One characteristic of the H–D model is that shear stress does not exceed τmax. The value of τmax can be calculated, on the basis of γ_r and G_0, from Eq. (8.1.6.2). The H–D model has two parameters, G_0 and γ_r. Since, however, G_0 is the modulus of elasticity, it is only γ_r that can be used to adjust nonlinear behavior. The parameter γ_r is called "reference strain," and common practice is to use the shear strain that makes $G/G_0 = 0.5$ hold true as reference strain for nonlinear dynamic analysis of ground. If, however, large ground motion is made to act from the base rock, analytically obtained strains may become very large and the acceleration response waves at ground surface may become very small. This is because the H–D model underestimates shear strength in the range beyond the reference strain (conversely, the R–O model overestimates shear strength in the range beyond the reference strain). In nonlinear dynamic analysis of ground, if such phenomena can be seen in analytical results when the H–D model is used, the reference strain is adjusted. For example, the H–D model can be prevented from underestimating shear stress by changing from the value of reference strain at $G/G_0 = 0.5$ to the value at $G/G_0 = 0.4$. In the analysis considered here, the maximum value of the acceleration response spectrum of input ground motion was as large as 1,000 gal, and it was thought likely that the shear strain in the ground would be large. A nonlinear dynamic analysis of the ground was conducted, therefore, by using the shear strain that makes $G/G_0 = 0.4$ hold true as the value of the reference strain γ_r, and the ground strain obtained from the analysis was checked. After that, for layers with larger strains, nonlinear dynamic analyses were conducted by using the shear strain that makes $G/G_0 = 0.3$ or $G/G_0 = 0.25$ hold true as the value of the reference strain γ_r. Finally, validity of the analysis was verified by checking whether the analytically determined shear strains stayed within the range of strain (2–3%) that can be traced in response analysis.

$$\tau = \frac{G_0 \gamma}{1 + \gamma / \gamma_r} \quad \cdots\cdots (8.1.6.1)$$

$$\tau_{\max} = G_0 \gamma_r \quad \cdots\cdots (8.1.6.2)$$

$$\gamma_r = \begin{cases} ① strain\ at\ G/G_0 = 0.5 \\ for\ \gamma_{\max} = \text{large strain} \\ ② strain\ at\ G/G_0 < 0.5 \end{cases} \quad \cdots\cdots (8.1.6.3)$$

G_0 : Initial shear modulus
τ : Shear stress
γ : Shear strain

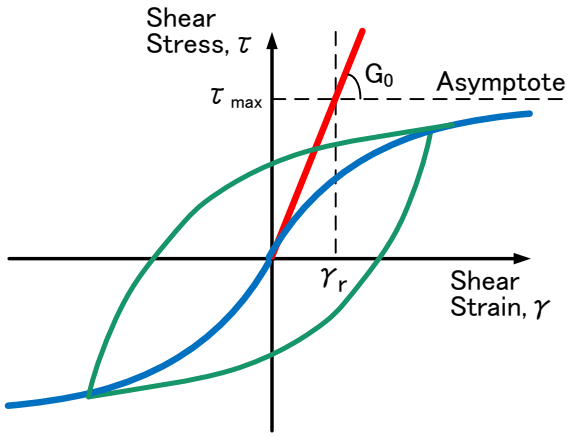


Figure 8.1.6-2 H-D Model (Hardin and Drnevich)

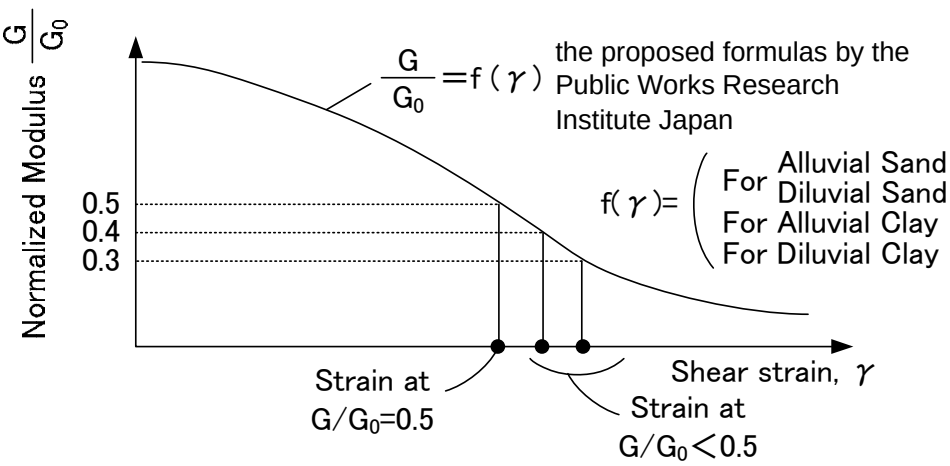


Figure 8.1.6-3 Relationship between Strain Dependence of Shear Modulus and γ_r

8.1.7 Analysis Results

(1) STEP1 : Evaluation of Earthquake Ground Motion

Figure 8.1.7-1 shows a flowchart for the development of earthquake ground motion matching the target spectrum. Figure 8.1.7-2 compares the acceleration response spectra of EQ1 ground motions developed and the target acceleration responses and shows the acceleration waveforms of the ground motions (Other EQs compatible to target spectrum are in Appendix 2-C(1)).

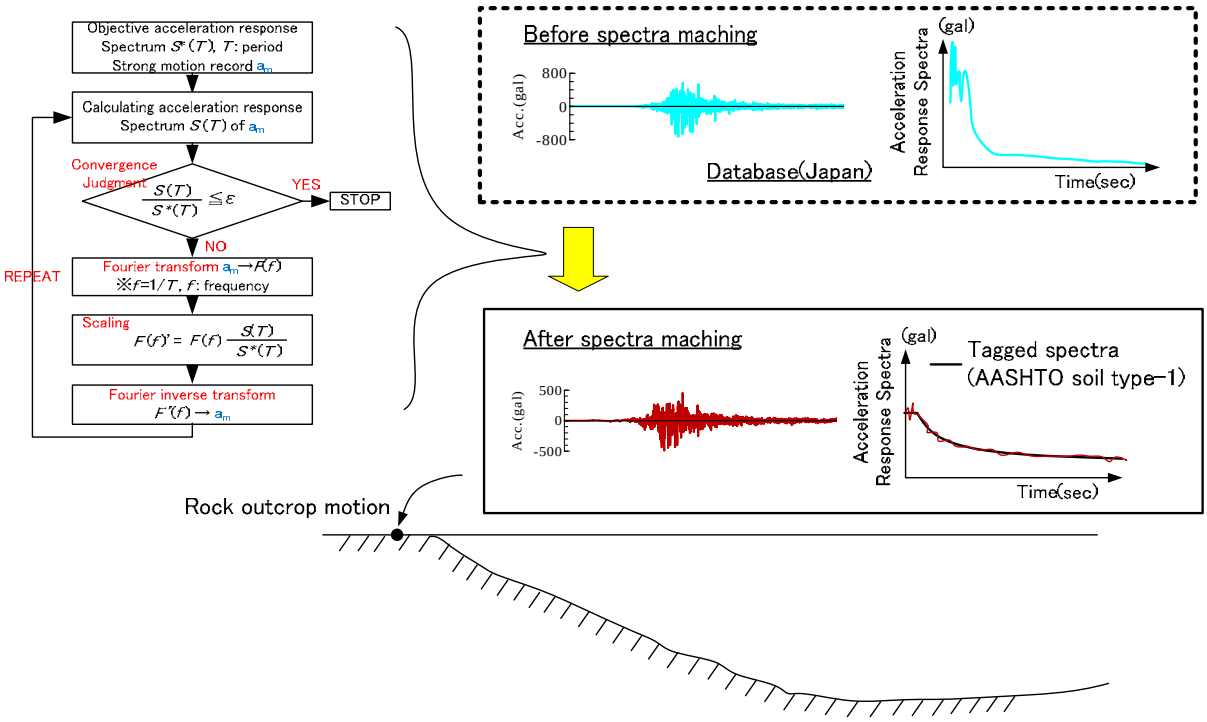


Figure 8.1.7-1 Generation of Earthquake Ground Motion Matching the Target Spectrum

In Figure 8.1.7-2 and the other figures in appendix2-C(1), the correspondence between target spectra and periods is indicated by vertical lines. Since this project aims to determine design earthquake ground motions for bridges, in view of the range of natural periods of bridges, periods of 0.05 sec to 5.0 sec were considered as periods to be matched to the target spectra. At the initial stage of matching, matching work was carried out at time steps of Δt = 0.1 sec by equally dividing the entire period range from 0.05 set to 5.0 sec. Since, however, spectrum values for periods shorter than 1 sec deviated considerably from the target spectra, the time step for matching period calculation was changed to Δt = 0.02 sec for natural periods shorter than 1 sec as shown in the figures so that matching work could be carried out in finer time steps. The figures show the matching results obtained at Δt = 0.02 sec. The light blue lines show the acceleration waveform and acceleration response spectrum of the pre-matching acceleration waveform. The red lines show the acceleration waveform and acceleration response spectrum obtained as a result of the matching process. As shown, the superposition of the acceleration response spectrum of the post-matching acceleration waveform and AASHTO's target spectrum represented by a black line gives the impression that the degree of matching between the acceleration response spectrum of the post-matching acceleration waveform and the target spectrum is not very good in the shorter period range. The reason for this is that although the acceleration response spectrum was calculated at fine time steps, the spectrum values for even shorter periods for which matching had not been done showed values that somewhat deviate from the target spectrum. In the ground analysis described in the next chapter, the matched waveforms developed as described above are used as inputs. Since, however, the goal is to evaluate the effect of longer-period ground motion on ground response rather than the effect of shorter-period (shorter than about 0.3 sec) ground motion for which the degree of matching to the target spectrum is not good, further matching at even finer time steps is not conducted here. Another reason why further matching at finer time steps is not conducted is that if further matching is done at finer period calculation time steps, post-matching waveforms can become unnaturally shaped.

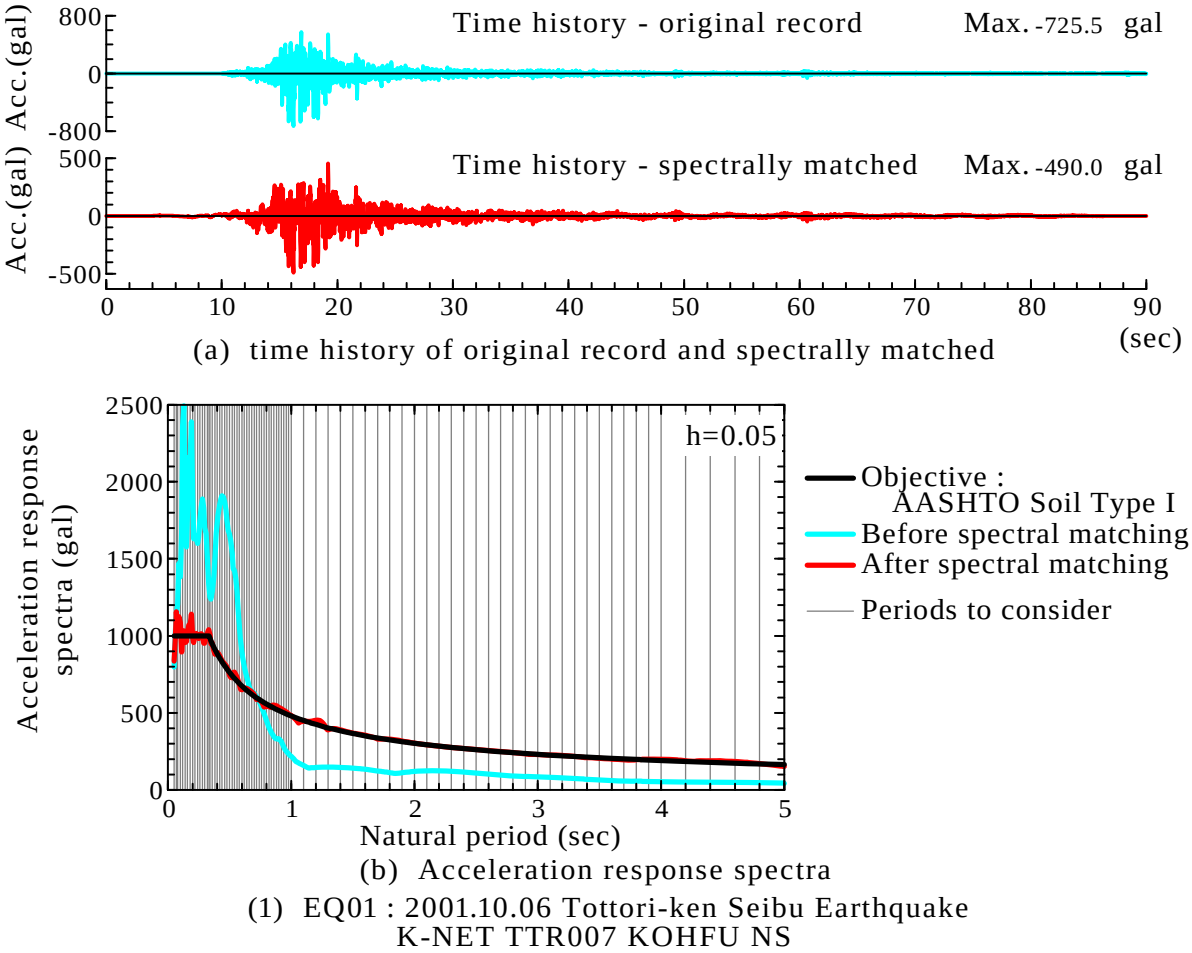


Figure 8.1.7-2 Compatible to Target Spectrum (Typical Conclusion: EQ1)

(2) STEP2 : Ground Motion Analysis

Figure 8.1.7-3 shows the response values of interest and the corresponding location of interest in the response analysis of ground at each site. Under consideration here are horizontal distributions of maximum acceleration at different depths, displacement distribution, shear strain in each layer, shear stress, shear stress–shear strain response history, and the maximum acceleration at ground surface.

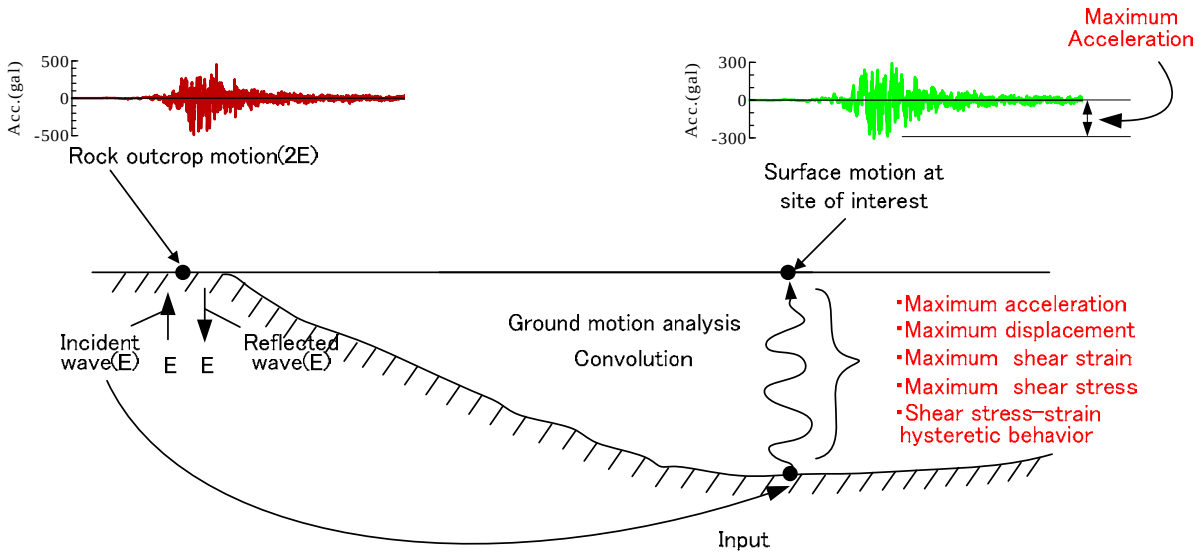


Figure 8.1.7-3 Response Values and Location of Interest

1) Ground Response History

This section explains the response history of the shear stress–shear strain relationship for ground. To explain ground behavior, Site No. 4, where ground is soft, and Site No. 1, where ground is moderately firm, are considered as examples. Figure 8.1.7-4 shows the response history of the shear stress–shear strain relationship for each layer of soft ground (Site No. 4). This result was obtained by using the EQ1 ground motion as an input at Site No. 4. As shown, in the ground at Site 4, the AC layer at depths of 7 m to 25 m shows hysteresis loops, indicating nonlinear behavior. The shallow AS layer at depths of 0 m to 7 m and the DC layer near the base rock at depths of 25 m to 33 m show linear shear stress–shear strain relationships, indicating linear behavior. It is thought likely that the behavior in the shallow AS layer was linear because ground shear stress increases with depth. It is also thought likely that the behavior in the DC layer was linear because the shear wave velocity VS is higher than in the AC layer and therefore the shear modulus and shear strength are high.

Figure 8.1.7-5 shows the response history of the shear stress–shear strain relationship for each layer in moderately firm ground (Site No. 1). The AS and AC layers at depths not greater than 7 m show small responses and linear behavior. This tendency is similar to the tendency shown by the AS layer at depths of 0 m to 7 m in the case where the EQ1 ground motion is input at Site No. 4, where ground is soft. The AS and AC layers at depths greater than 7 m show hysteresis loops, indicating nonlinear behavior. Nonlinear behavior is more pronounced in the AS layer at depths of 7 m to 8 m than in the underlying AC layer at depths of 8 m to 16 m, where the shear wave velocity VS is low. The AS layer at depths of 7 m to 8 m is sandwiched between the overlying and underlying AS layers with different shear wave velocities (VS), that is, sandwiched between layers having different stiffnesses and strengths. Consequently, it is thought, relative displacement became large so as to increase the nonlinearity of behavior.

Figure 8.1.7-6 shows the results obtained in the case where the EQ13 ground motion is input at a moderately firm ground site (Site No. 1). As shown, responses are greater than in the case where the EQ1 ground motion is input shown in Figure 8.1.7-5. The AS layer at depths of 7 m to 8 m and the AS layer at depths of 16 m to 18 m show large hysteresis loops, indicating higher degrees of nonlinearity. As can be seen from Figure 8.1.7-5 and Figure 8.1.7-6, ground response varies depending on the phase of input ground motion. Ground response tends to show higher degrees of nonlinearity in layers in which stiffness varies considerably in the direction of depth and deep layers in which shear stress is large.

2) Ground Maximum Response Value Distributions

Figure 8.1.7-7 shows maximum response value distributions in the direction of depth at Site No. 1, where ground is soft. Figure 8.1.7-8 shows maximum response value distributions at Sites No. 1, where ground is moderately firm. The knowledge gained from the maximum response value distributions is described below. (For other Sites, see Appendix2-C(2).)

a) Maximum acceleration

The maximum acceleration shows two types of tendency: it either tends to decrease as depth decreases or tends to remain almost unchanged. In cases where shear strain in the ground is large and the degree of nonlinearity of ground is high, the maximum acceleration at ground surface tends to be smaller than the maximum acceleration of ground motion input at the base rock. In cases where shear strain in the ground is small and the degree of nonlinearity of ground is low or in cases where shear strain is large but a layer near the ground surface behaves mainly nonlinearly, the maximum acceleration at ground surface and the maximum acceleration of ground motion input at the base rock show more or less similar values. The reason for this is that in cases where the ground motion input at the base rock is large, shear strain in the ground becomes large so that the response shear force τ approaches the shear strength τmax and therefore the acceleration acting on the ground shows relatively large values. In cases where the ground motion input at the base rock is small, the ground shows behavior close to elastic response. Consequently, the ground motion input at the base rock is amplified so that the maximum acceleration at ground surface becomes larger than the maximum acceleration of the ground motion input at the base rock.

b) Maximum relative displacement

At Sites No. 1, No. 2 and No. 5, where ground is soft, shear strain in about-5-m-thick layers is large, and shear displacements at ground surface of about 15 cm occur. At Site No. 6, shear strain is smaller than at No. 1, No. 2 and No. 5, but shear displacements at ground surface of about 15 cm occur because the region in which a shear strain of about 1% occurs is large. At No. 3, as at Sites No. 1, No. 2, and No. 5, shear strain in the layer having a thickness of about 5 m is large, but the shear displacement at ground surface stays within about 10 cm because shear strain is somewhat smaller than at the three sites mentioned above. At Site No. 4, shear strain is smaller than at other sites, and shear displacement at ground surface stays within about 10 cm.

At Sites No. 1, No. 2 and No. 5, where ground is moderately firm, shear displacement at ground surface is within about 5 cm because shear strain in the ground is small. At No. 3, shear strain near the ground surface is large, but because the layer is thin, shear displacement at ground surface stays within about 5 cm. At No. 4 and No. 6, shear displacement at ground surface is as large as about 10 to 15 cm because the layer in which ground shear strain is large is thick.

Shear wave velocity Vs in soft ground is lower than that in moderately firm ground. Soft ground, therefore, is less stiff than moderately firm ground, and the thickness of layers from the ground surface to the base rock in soft ground is greater than that in moderately firm ground. Consequently, the shear displacement at the surface of soft ground is larger than the shear displacement at the surface of moderately firm ground. It seems that cases in which shear displacement at ground surface is large can be broadly classified into two types: cases in which shear strain in a small number of layers is large and cases in which shear strain in many layers is large.

c) Maximum shear strain

It can be seen that at Sites No. 1, No 2, No. 4 and No. 5, where ground is soft, shear strain tends to be large in layers in which Vs is low. This is because the dynamic shear modulus G_D of the ground increases in proportion to Vs as shown in Eq. (8.1.7-1). Since the shear modulus of a low-Vs layer is low, if the same amount of shear stress acts, shear strain becomes larger than in other high-shear-modulus layers as shown in Eq. (8.1.7-2).

$$G_D = \frac{\gamma_t}{g} V_S^2 \quad (8.1.7\text{-}1)$$

$$\tau = G_D \gamma \quad (8.1.7\text{-}2)$$

Where,

G_D : Dynamic shear modulus of the ground, γ_t: unit weight of ground, g : Acceleration of gravity, V_s :Shear elastic wave velocity of the ground, τ : Shear stress, γ : Shear strain

At Sites No. 3 and No. 6, at depths where soil type changes abruptly as in the case of an AC–AS–AC formation, strain in the intermediate layer (i.e., the As layer) sandwiched by two layers of different soil type is large.

At Sites No. 1 and No. 3, where ground is moderately firm, shear strain in low-Vs layers is large as in soft ground. At Site No. 6, as in soft ground, at depths where soil type changes abruptly as in the case of an AC–AS–AC formation, strain in the intermediate layer (i.e., the As layer) sandwiched by two layers of different soil type is large. At Sites No. 2, No. 4 and No. 5, large shear strain occurs in a layer overlying the base rock where large shear stress occurs.

The strain distributions in soft ground and moderately firm ground seem to indicate that layers in which large shear strain occurs can be broadly classified into three types: (1) a low-Vs layer, (2) a layer such as a layer in an AC–AS–AC formation in which soil type changes abruptly and (3) a layer overlying the base rock where large shear stress occurs.

d) Maximum shear stress

The distributions of the maximum shear stresses in soft ground and moderately firm ground indicate that shear stress increases with ground depth. This is because a deep layer must resist the horizontal force acting on all overlying layers. Comparison of the maximum shear stress distributions in soft ground and moderately firm ground reveals that in moderately firm ground, the relationship between depth and shear stress is more or less linear, while in soft ground, the depth–shear stress relationship is not linear. The reason for this is that the influence of ground plasticization is greater in soft ground than in moderately firm ground, and, as mentioned earlier, shear strain becomes large in soft ground so that the response shear force τ acting on each layer approaches the shear stress τ_{max} of each layer.

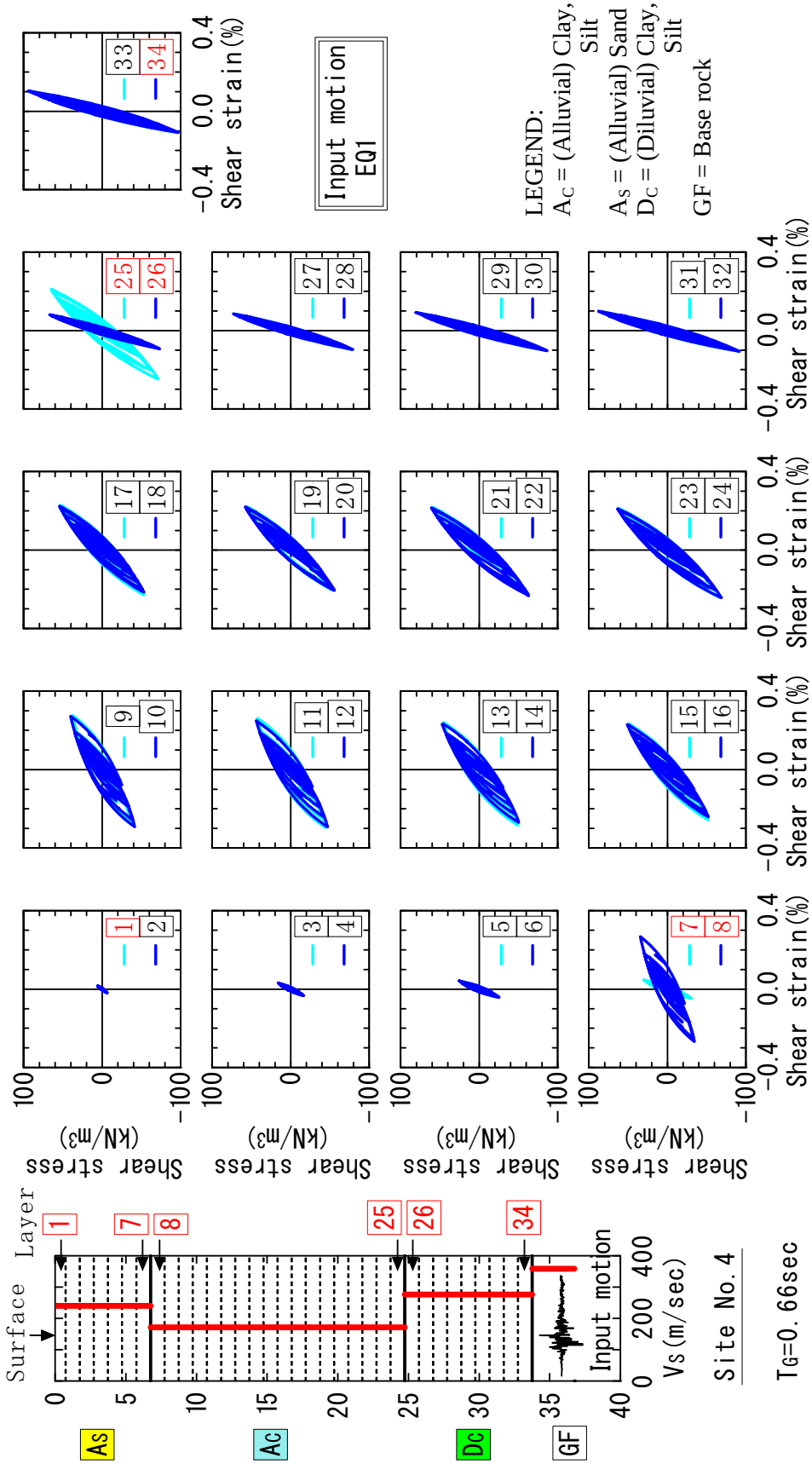


Figure 8.1.7-4 Shear Stress-Strain Hysteretic Behavior of Layers under EQ1 (Soft Ground, site No.4)

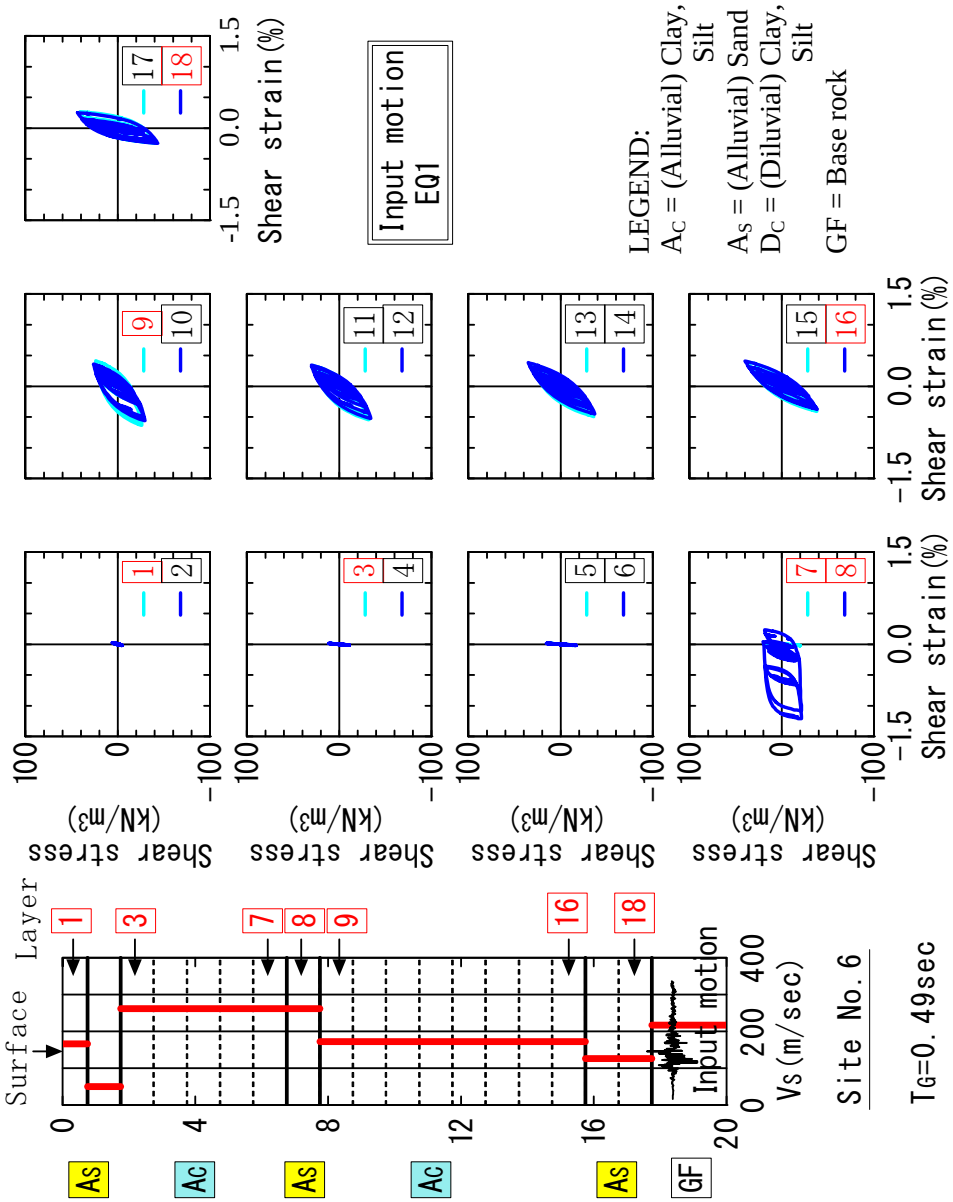


Figure 8.1.7-5 Shear Stress-strain Hysteretic Behavior of Layers under EQ1 (Moderate Firm Ground, Site No.1)

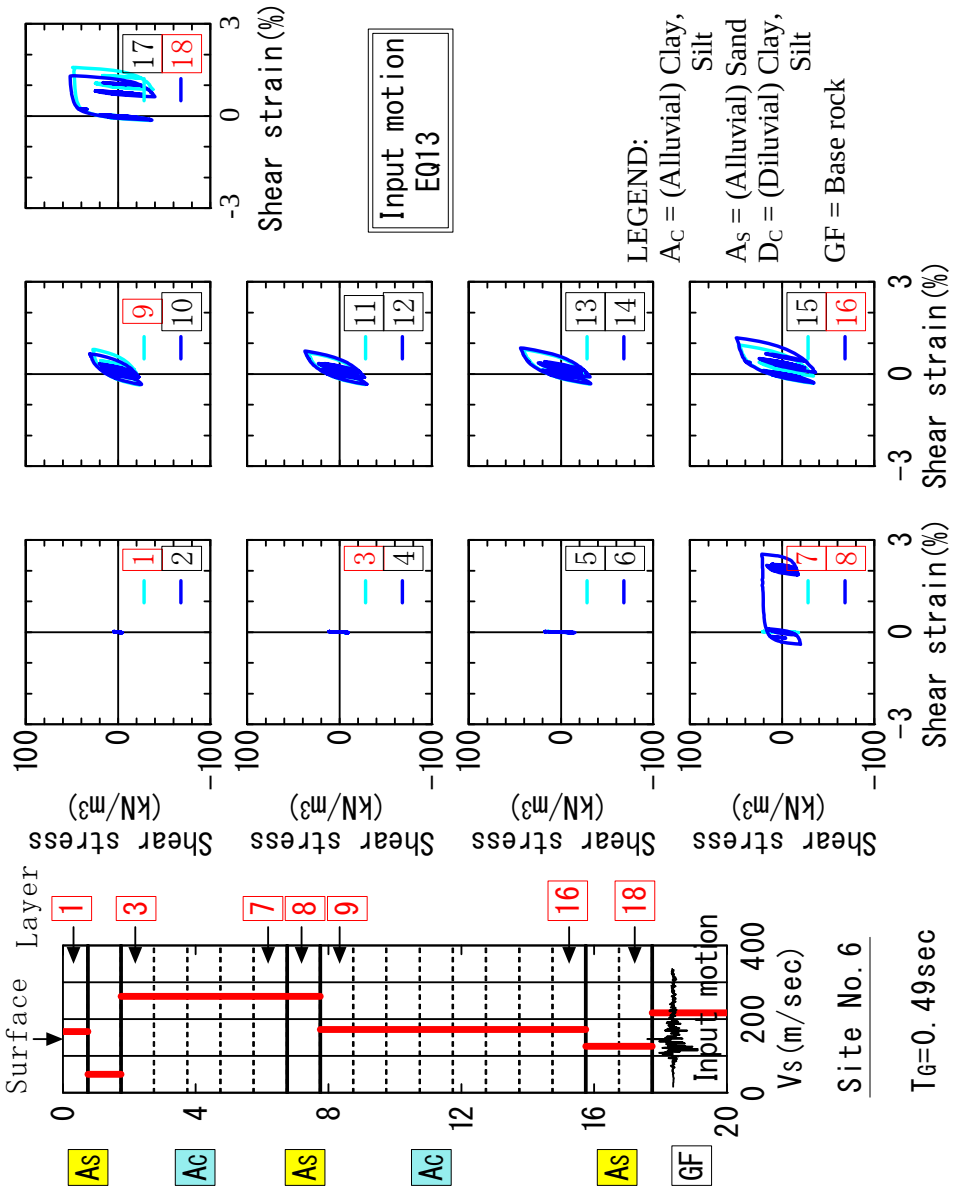


Figure 8.1.7-6 Shear Stress-Strain Hysteretic Behavior of Layers under EQ13 (Moderate Firm Ground, Site No.1)

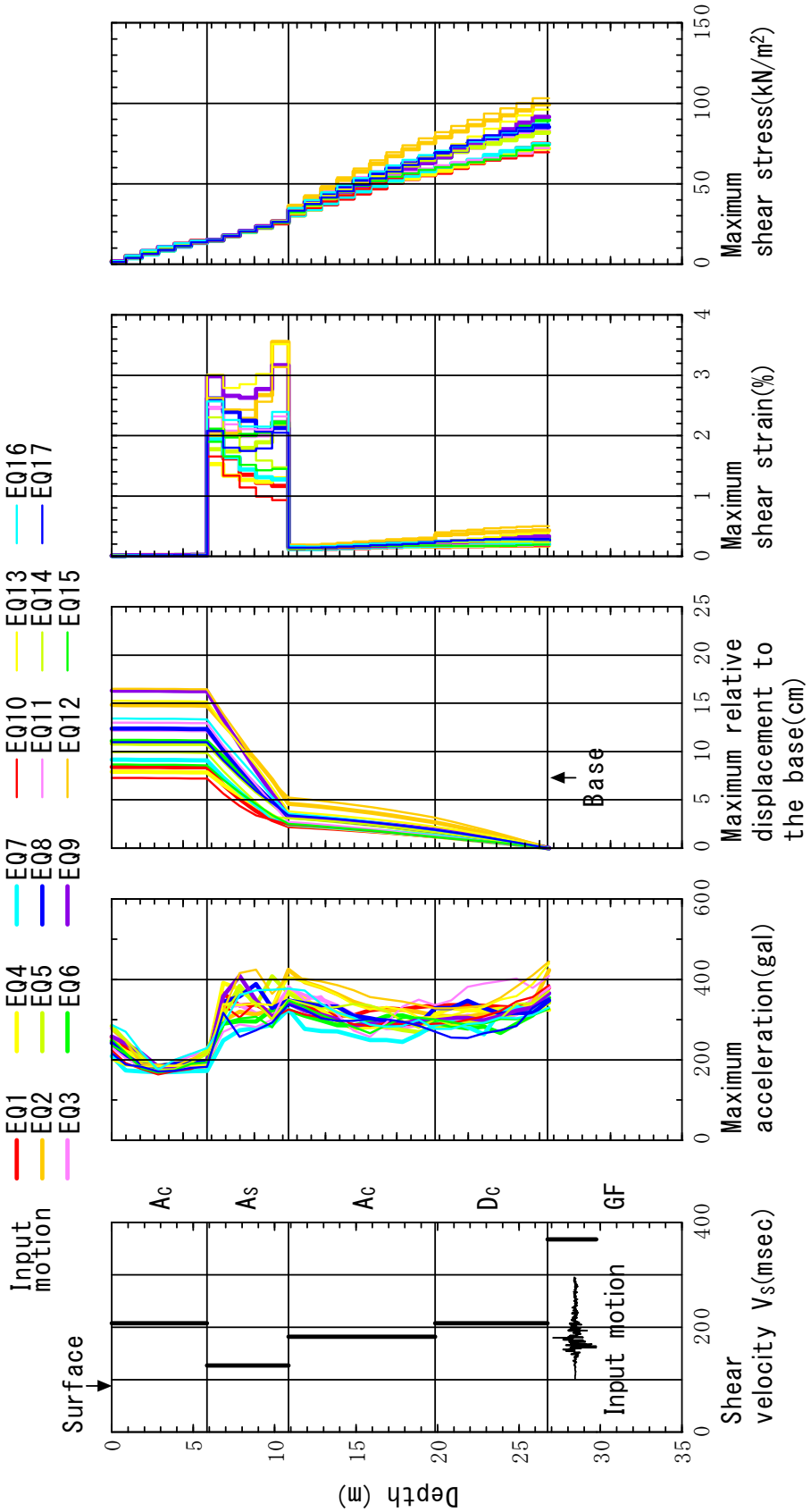


Figure 8.1.7-7 Maximum Acceleration, Maximum Displacement, Maximum Shear Strain and Maximum Shear Stress at Different Layers (Soft Ground, Site No.1)

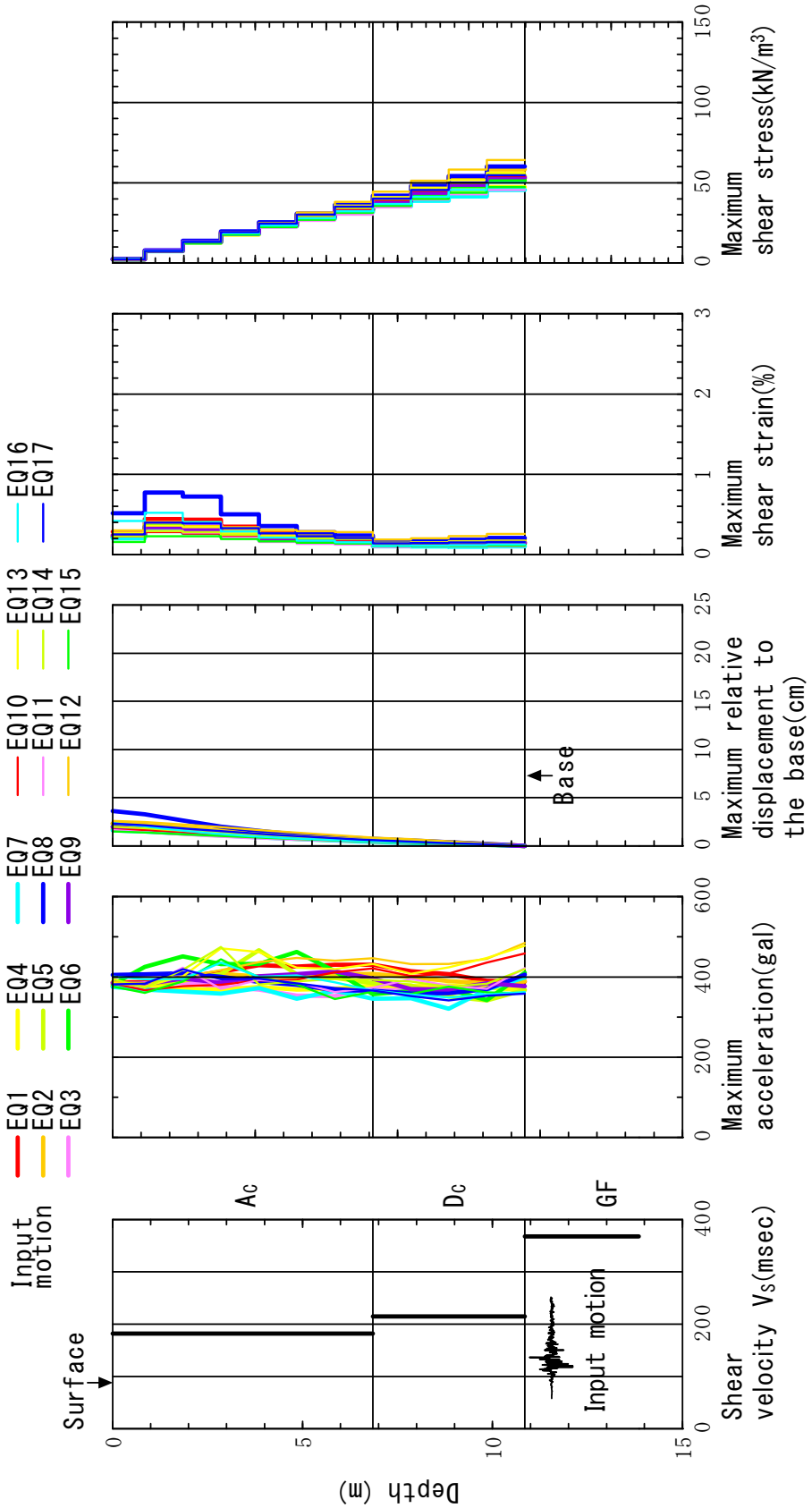


Figure 8.1.7-8 Maximum Acceleration, Maximum Displacement, Maximum Shear Strain and Maximum Shear Stress at Different Layers (Moderate Firm Ground, Site No.1)

(3) Maximum Acceleration and Acceleration Amplification Factor

1) Maximum Acceleration

Figure 8.1.7-10 and Figure 8.1.7-11 compare the maximum accelerations at ground surface of soft ground and moderately firm ground shown in Figure 8.1.7-9 obtained from ground analysis with the maximum accelerations of outcrop motion (incident wave of outcrop motion is input at the base rock in soft ground and moderately firm ground) defined at the rock outcrop at the ground surface. The knowledge gained from Figure 8.1.7-10 and Figure 8.1.7-11 are shown below.

- As ground shear strain reaches a certain level, the response shear force τ approaches the shear strength τ_{max}. As a result, acceleration at ground surface becomes constant.
- For the reason mentioned above, the acceleration at the surface of soft ground or moderately firm ground tends to be smaller than that of outcrop motion.
- The shear modulus and shear strength of moderately firm ground are higher than those of soft ground. Response shear strain in moderately firm ground, therefore, tends to be smaller than that in soft ground, and the surface acceleration of moderately firm ground tends to be larger than that of soft ground.

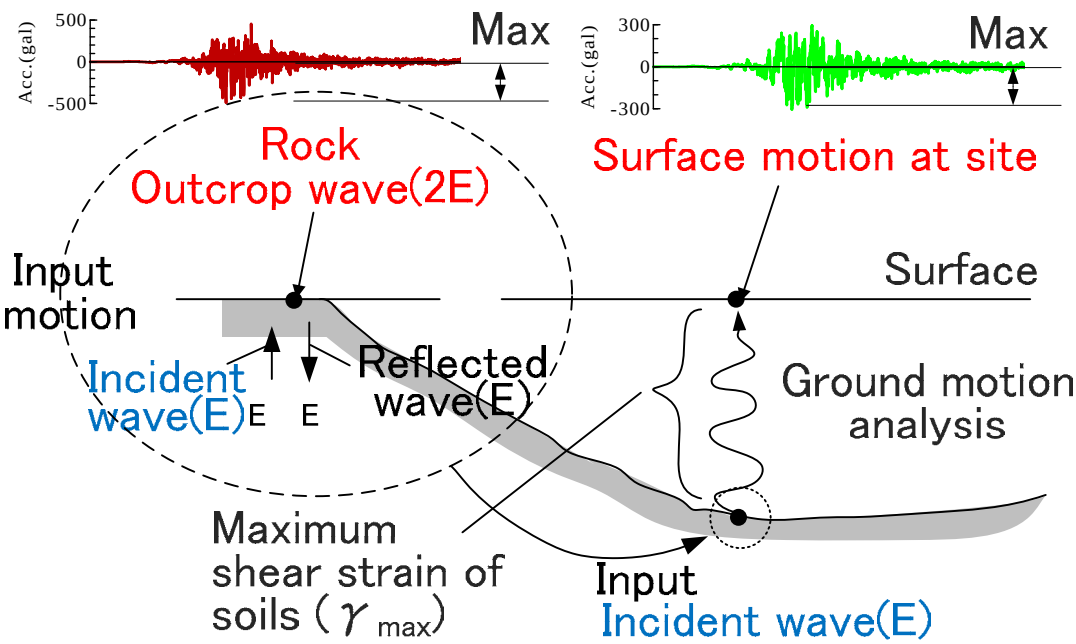


Figure 8.1.7-9 Comparison of Maximum Surface Accelerations of Soft Ground and Moderately Firm Ground and Maximum Acceleration at Outcrop Motion Defined at Rock Outcrop at Ground Surface

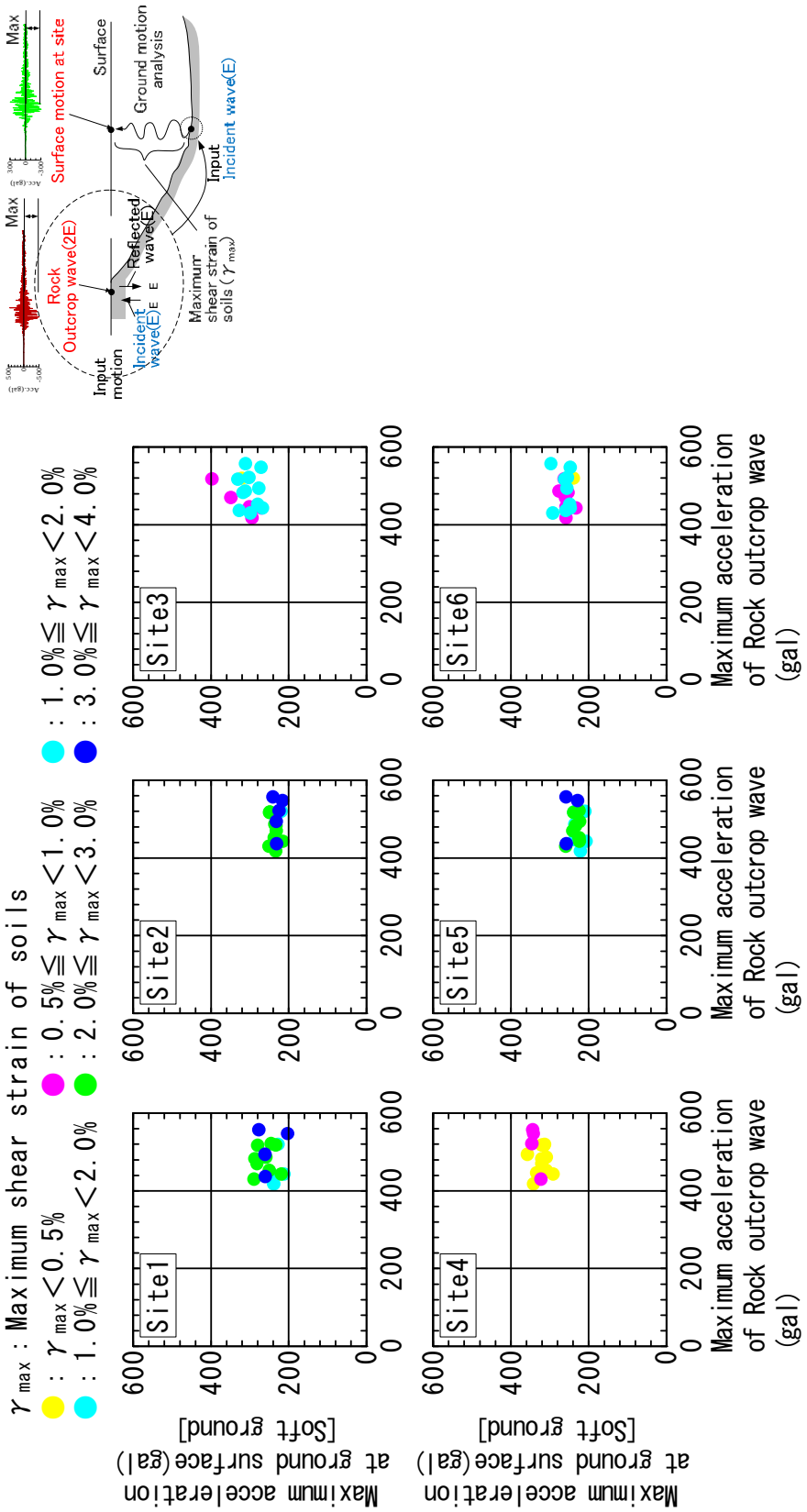


Figure 8.1.7-10 Comparison of Maximum Surface Acceleration of Soft Ground and Maximum Acceleration of Outcrop Motion Defined at Rock Outcrop at Ground Surface

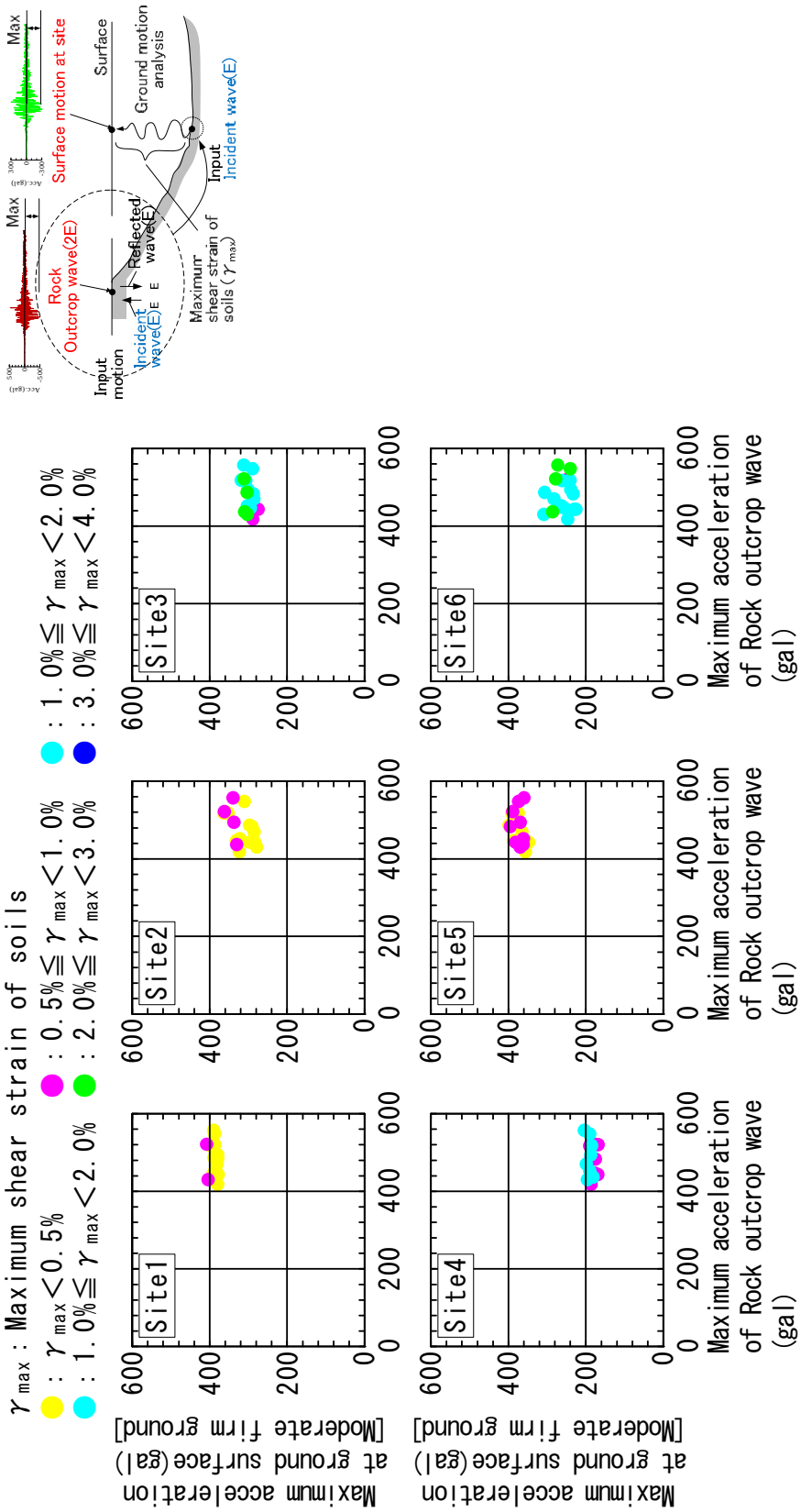


Figure 8.1.7-11 Comparison of Maximum Surface Acceleration of Moderate Firm Ground and Maximum Acceleration of Outcrop Motion Defined at Rock Outcrop at Ground Surface

2) Estimation of Acceleration Amplification Factor

The ratio between the maximum surface accelerations of soft ground and moderately firm ground shown in Figure 8.1.7-12 obtained from ground analysis and the maximum acceleration of the incident wave input at the base rock was calculated. Then, evaluation was made to determine to what extent the maximum amplitude of acceleration is amplified while the incident wave propagates from the base rock through the surface strata to the ground surface. The amplification factor formula is shown below.

Acceleration amplification factor G_S

$$G_S = \frac{A_{CC(S)}}{A_{CC(B)}} \qquad (8.1.7\text{-}3)$$

Where,
Acc(S) : Maximum acceleration at surface
Acc(B) : Maximum acceleration of incident wave

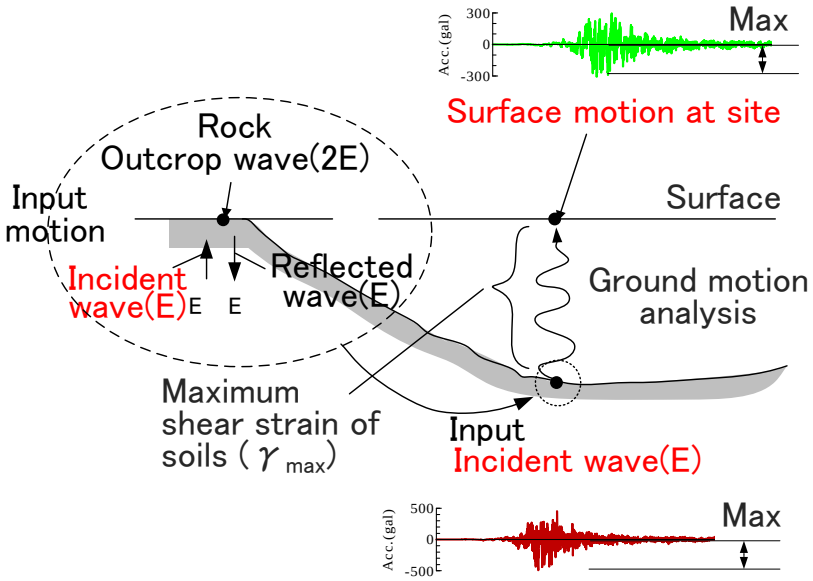


Figure 8.1.7-12 Estimation of Acceleration Amplification Factor

Figure 8.1.7-13 and Figure 8.1.7-14 show the acceleration amplification factors for soft ground and moderately firm ground calculated from Eq. (8.1.7). Table 8.1.7-1 shows the acceleration amplification factors estimated, like the acceleration amplification factors shown in Figure 8.1.7-13 and Figure 8.1.7-14, from the design horizontal seismic coefficients at ground surface for firm ground (JRA Soil Type I), moderately firm ground (JRA Soil Type II) and soft ground (JRA Soil Type III). The knowledge gained from Figure 8.1.7-13, Figure 8.1.7-14 and Table 8.1.7-1 are as follows:

- For soft ground, acceleration amplification factors of up to about 1.6 have been obtained.
- For moderately firm ground, acceleration amplification factors of up to about 1.9 have been obtained.
- If the maximum acceleration of the incident wave input at the base rock is small, shear strain in the surface layer of ground also becomes small so that ground response becomes closer to elastic response and the acceleration amplification factor becomes larger.
- The acceleration amplification factors obtained from ground analysis and the acceleration amplification factors estimated from JRA's design seismic coefficient at ground surface show fairly good agreement.

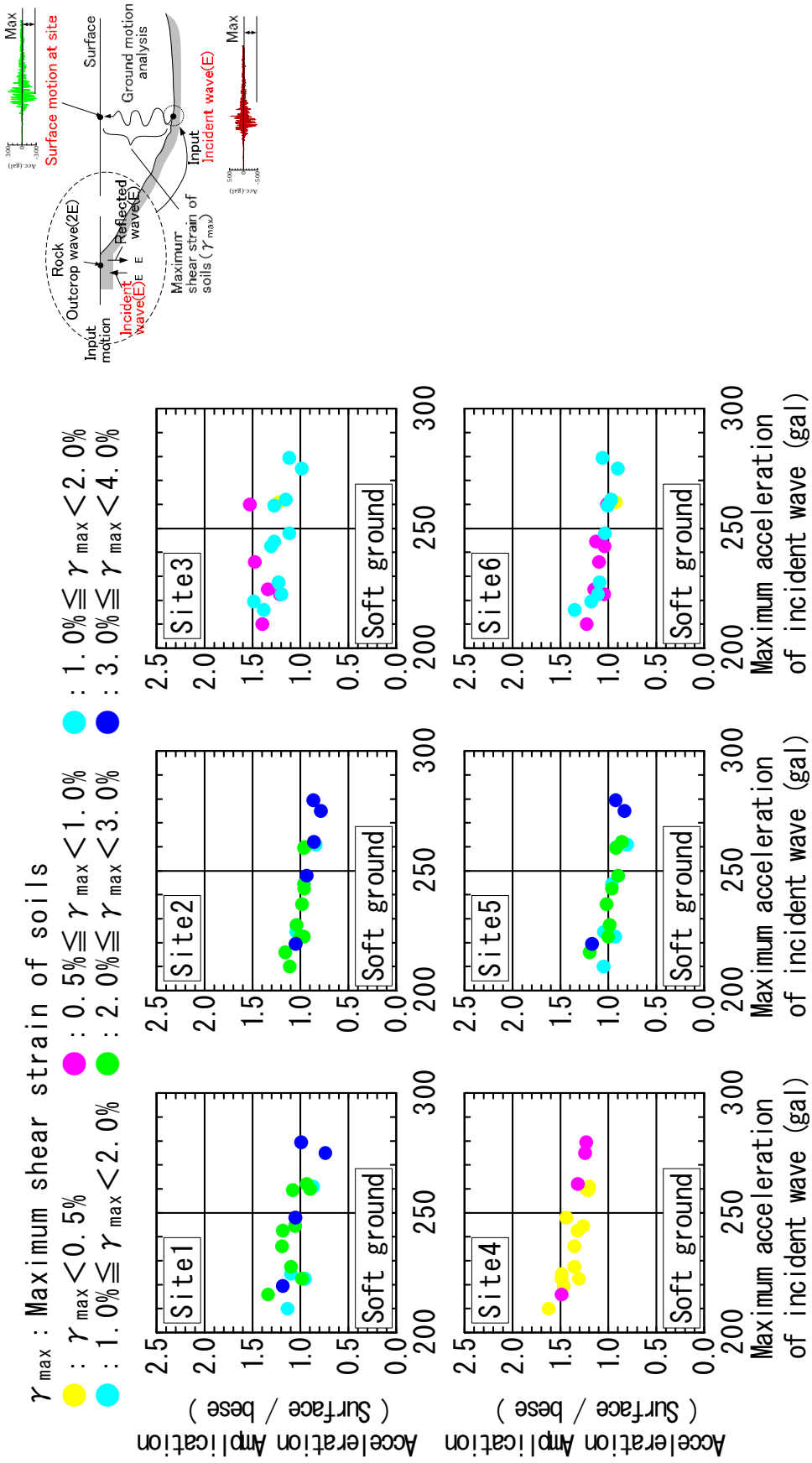


Figure 8.1.7-13 Acceleration Amplification Factor (Soft Ground)

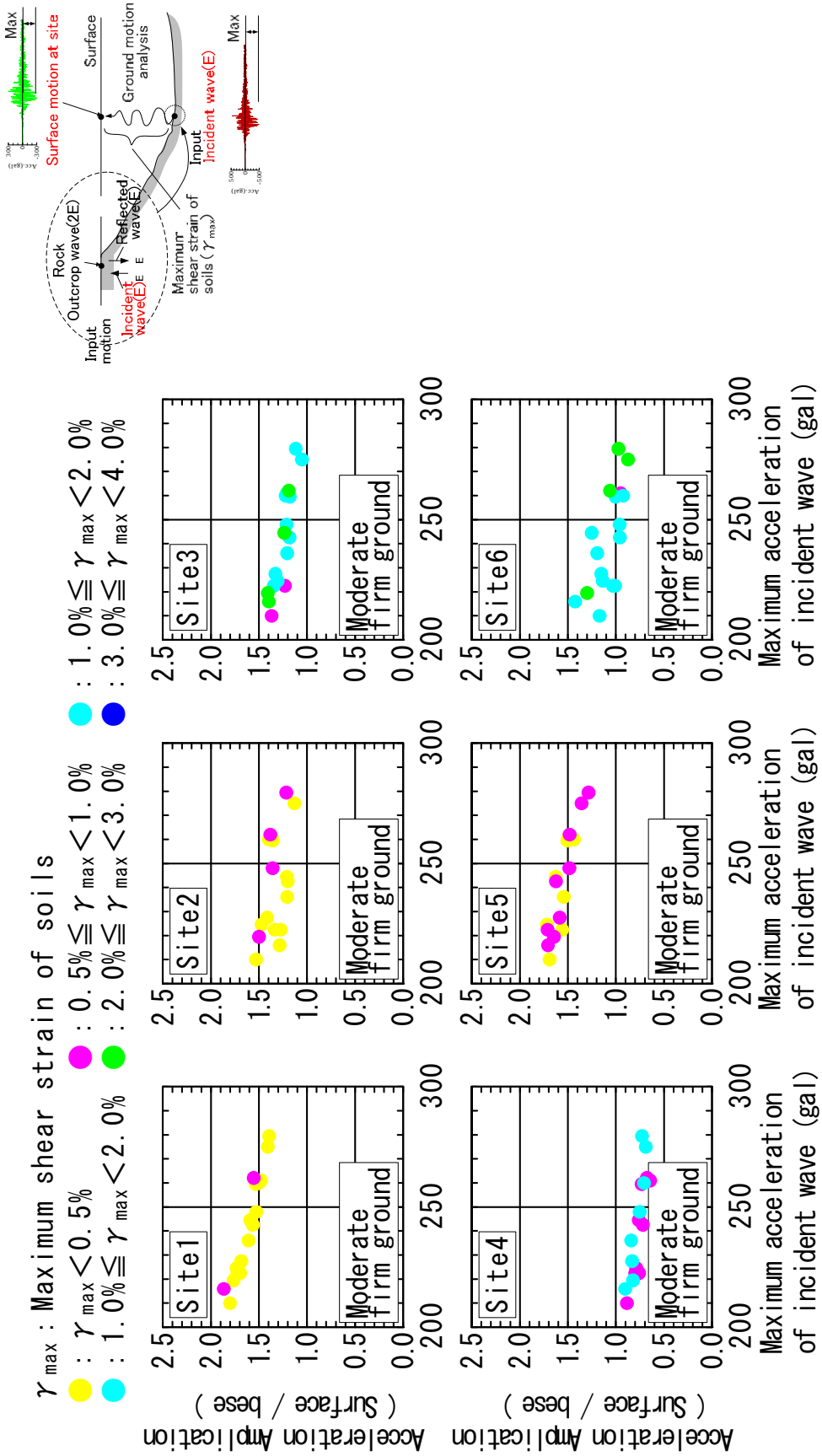


Figure 8.1.7-14 Acceleration Amplification Factor (Moderate Firm Ground)

Table 8.1.7-1 Comparison of Acceleration Amplification Factor

		AASHTO 2007	JRA Type I 2002	JRA Type I 2012	JRA Type II 2012	Ground motion analysis
$A_{CC(B)}$	Firm ground (2E) (E)	k_{hg}[1)]=0.4 (0.2)	k_{hg}=0.3 (0.15)	k_{hg}=0.6 (0.3)	k_{hg}=0.8 (0.4)	
$A_{CC(S)}$	Moderate firm ground	k_{hg}=0.32	k_{hg}=0.35	k_{hg}=0.54	k_{hg}=0.7	
	Soft ground	k_{hg}=0.32	k_{hg}=0.4	k_{hg}=0.48	k_{hg}=0.6	
Amplification factor $A_{CC(S)}/A_{CC(B)}$		0.32 / 0.2 =1.6	0.4 / 0.15 =2.7	0.54 / 0.3 =1.8	0.6 / 0.4 =1.5	Moderate firm ground (Max 1.9) Soft ground (Max 1.6)

1)k_{hg} : Design horizontal seismic coefficient at the ground level for Level1 or Level2 Earthquake Ground Motion(Type I or Type II).

(4) Estimation of Spectral Amplification Factor

The ratio between the acceleration response spectra at ground surface of soft ground and moderately firm ground shown in Figure 8.1.7-15 obtained from ground analysis and the acceleration response spectra of outcrop motion (whose incident wave is input at the base rock of soft ground and moderately firm ground) defined at the rock outcrop at the ground surface was calculated for each period. Then, the amplification factor for the acceleration response spectra at ground surface of soft ground and moderately firm ground relative to the acceleration response spectra of the input ground motion (outcrop motion) was evaluated. In evaluating the acceleration response spectra at ground surface at each site, the average of the acceleration response spectra obtained from all input ground motions was calculated in view of the variability of acceleration response spectra at ground surface depending on the phase of the input ground motion. In order to prevent inherent characteristics from being lost as a result of averaging, the maximum values of acceleration response spectra at different sites were also evaluated. The amplification factor formula is shown below.

Spectral amplification factor $G_S(t)$

$$G_S(T) = \frac{S_S(T)}{S_B(T)} \quad \cdots\cdots (8.1.7\text{-}4)$$

$$0 \leqq T \leqq 4.0$$

Mean of $G_S(T)$

$$G_S(T)_{mean} = \frac{\sum_{EQ=1}^{EQ=17} G_S(T)}{17} \quad \cdots\cdots (8.1.7\text{-}5)$$

Max of $G_S(T)$

$$G_S(T) = \max\left(G(T)_{EQ=1\sim17}\right) \quad \cdots\cdots (8.1.7\text{-}6)$$

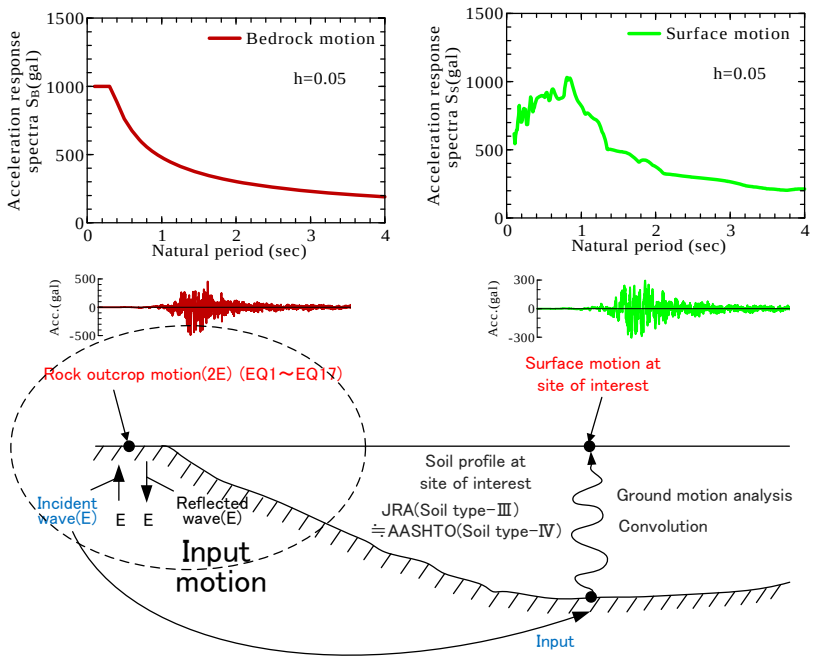


Figure 8.1.7-15 Estimation of Spectral Amplification Factor

Figure 8.1.7-17 and Figure 8.1.7-18 show the means and maximum values of the amplification factor determined at each site. Figure 3-1 to Figure 3-12 in Appendix2-C(3) show the amplification factors determined for different ground motions at different sites. The knowledge gained from Figure 8.1.7-17 and Figure 8.1.7-18 are shown below:

<Soft ground>

- The amplification factor takes the maximum value at around the first natural period determined from the initial soil stiffness.
- The reason why there are cases in which the maximum value occurs away from the first natural period is thought to be that the natural period has become longer because of ground plasticization.
- The maximum value of the amplification factor is around 2 at all sites excluding Site 1 and Site 5.
- Although the maximum value is smaller than 2 in some cases, amplification factor distributions show more or less similar patterns.

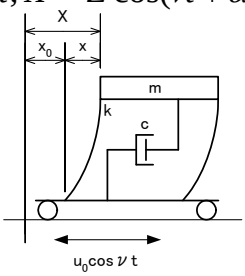
<Moderate firm ground>

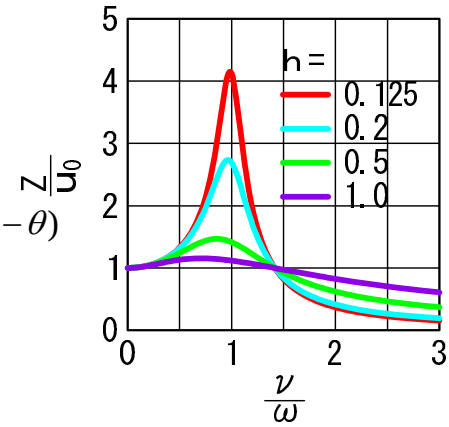
- Similar to soft ground sites, the amplification factor takes the maximum value at around the first natural period.
- Except at Site 1, where the ground is closest to rock, the amplification factor tends to become larger as the first natural period becomes longer.
- The maximum value of the amplification factor ranges from 1.5 to 2.0, showing values somewhat smaller than those at soft ground sites.
- Distribution tends to vary more widely than at soft ground sites depending on site-specific conditions.

Just for reference, resonance curves for the absolute displacement of a single-degree-of-freedom system excited by a sinusoidal displacement are shown below. In the case of elastic response, the amplification factor is as high as about 3 even in cases where a damping ratio of 20% is assumed. Care should be taken, however, in making a comparison with the results mentioned earlier because the horizontal axis of the graph shows frequency.

$$\frac{Z}{u_0} = \frac{\sqrt{1+(2h\frac{\nu}{\omega})^2}}{\sqrt{(1-\frac{\nu^2}{\omega^2})^2+(2h\frac{\nu}{\omega})^2}}$$

$$x_0 = u_0 \cos \nu t,\ X = Z\cos(\nu t + \alpha - \theta)$$





$$\omega = \sqrt{k/m}, 2h\omega = c/m$$

Figure 8.1.7-16 For Reference Only: Resonance Curves for Absolute Displacement of a Single-Degree-of-Freedom System Excited by Sinusoidal Displacement

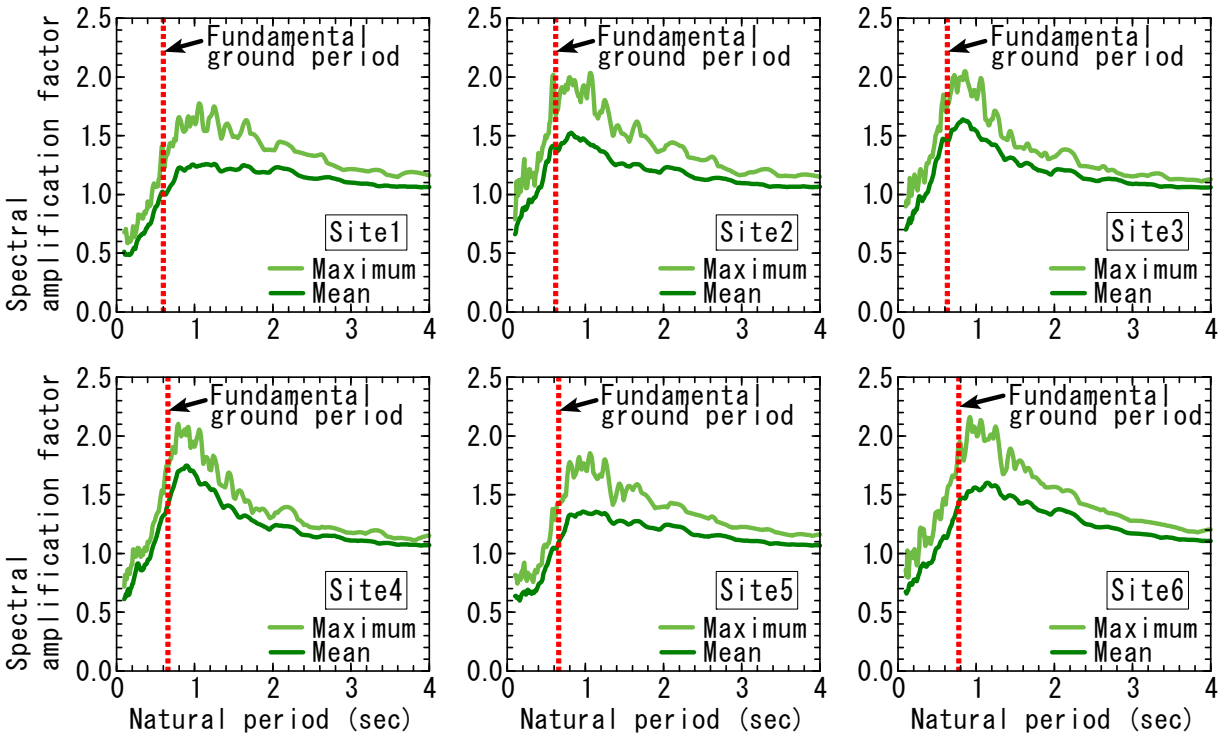


Figure 8.1.7-17 Spectral Amplification Factor (Soft Ground)

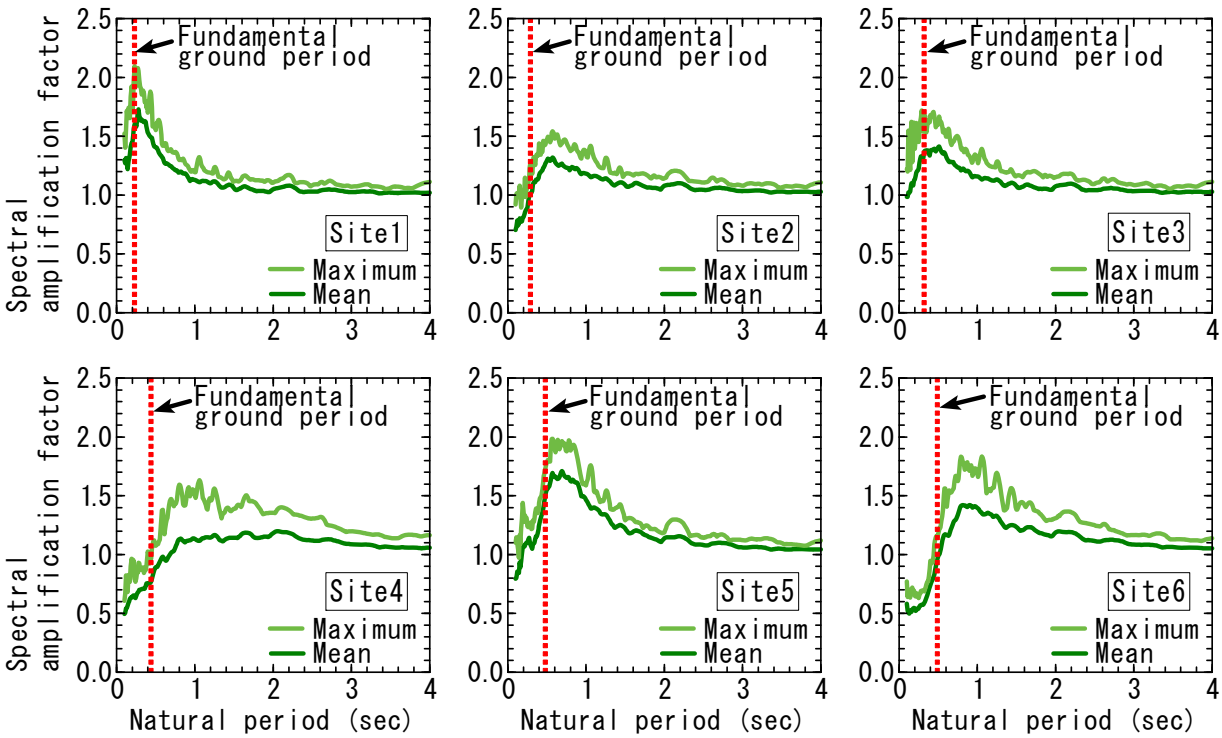


Figure 8.1.7-18 Spectral Amplification Factor (Moderate Firm Ground)

8.1.8 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications

(1) Comparison on the Shapes of Acceleration Response Spectra

As shown in Figure 8.1.8-1, the acceleration response spectra determined from the ground surface acceleration waveforms obtained from ground analysis and AASHTO's acceleration response spectra were compared. Figure 8.1.8-2, Figure 8.1.8-3 and figures in Appendix2-C(4) show the comparison results. Figure 8.1.8-2shows the results of the analyses conducted for Sites No. 1 (soft ground site). Figure 8.1.8-3 shows the results of the analyses conducted for Sites No. 1 (moderately firm ground site).

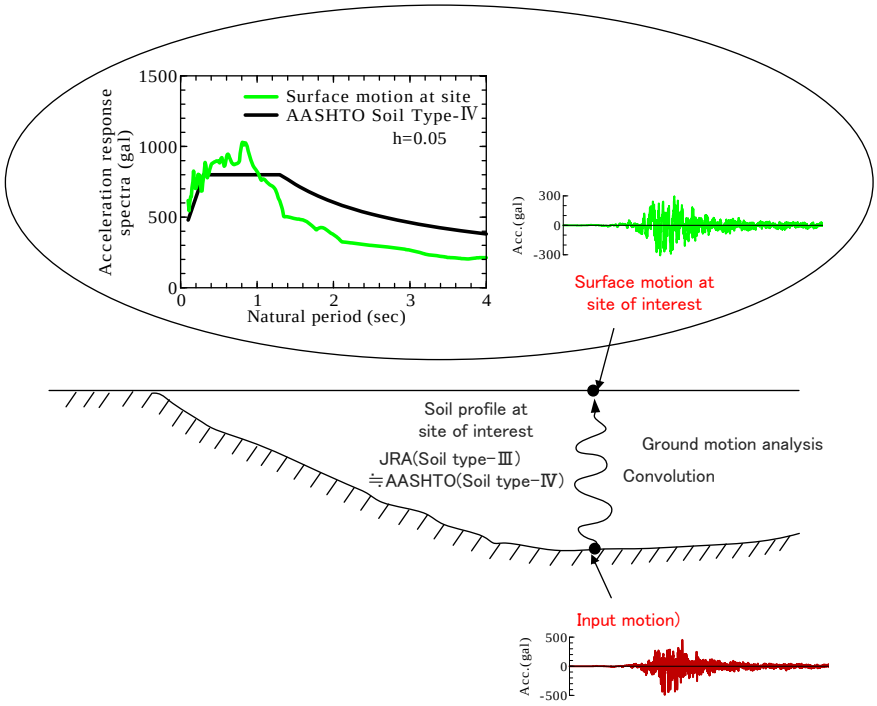


Figure 8.1.8-1 Response Values and Location of Interest

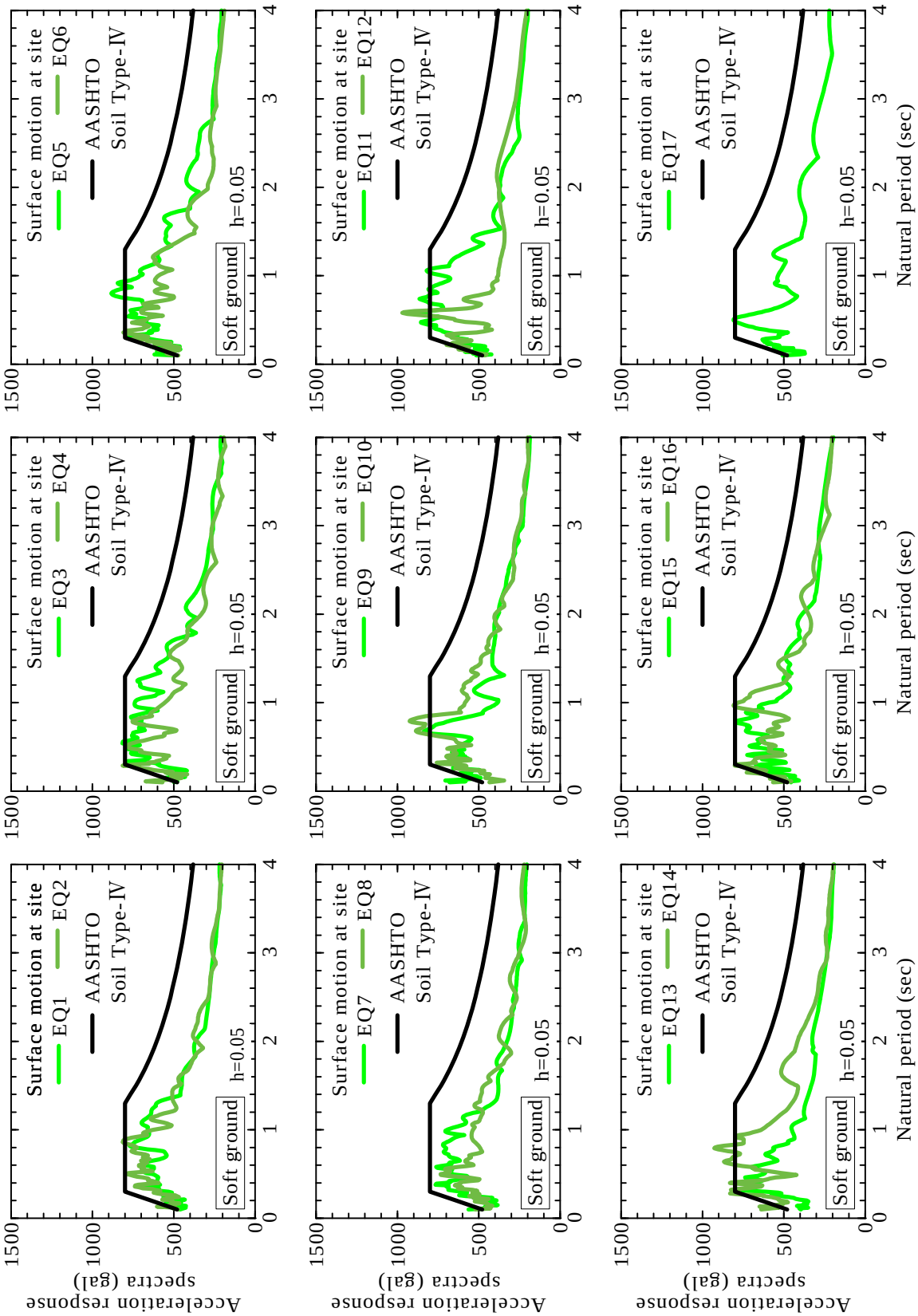


Figure 8.1.8-2 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft Ground: Site No.1)

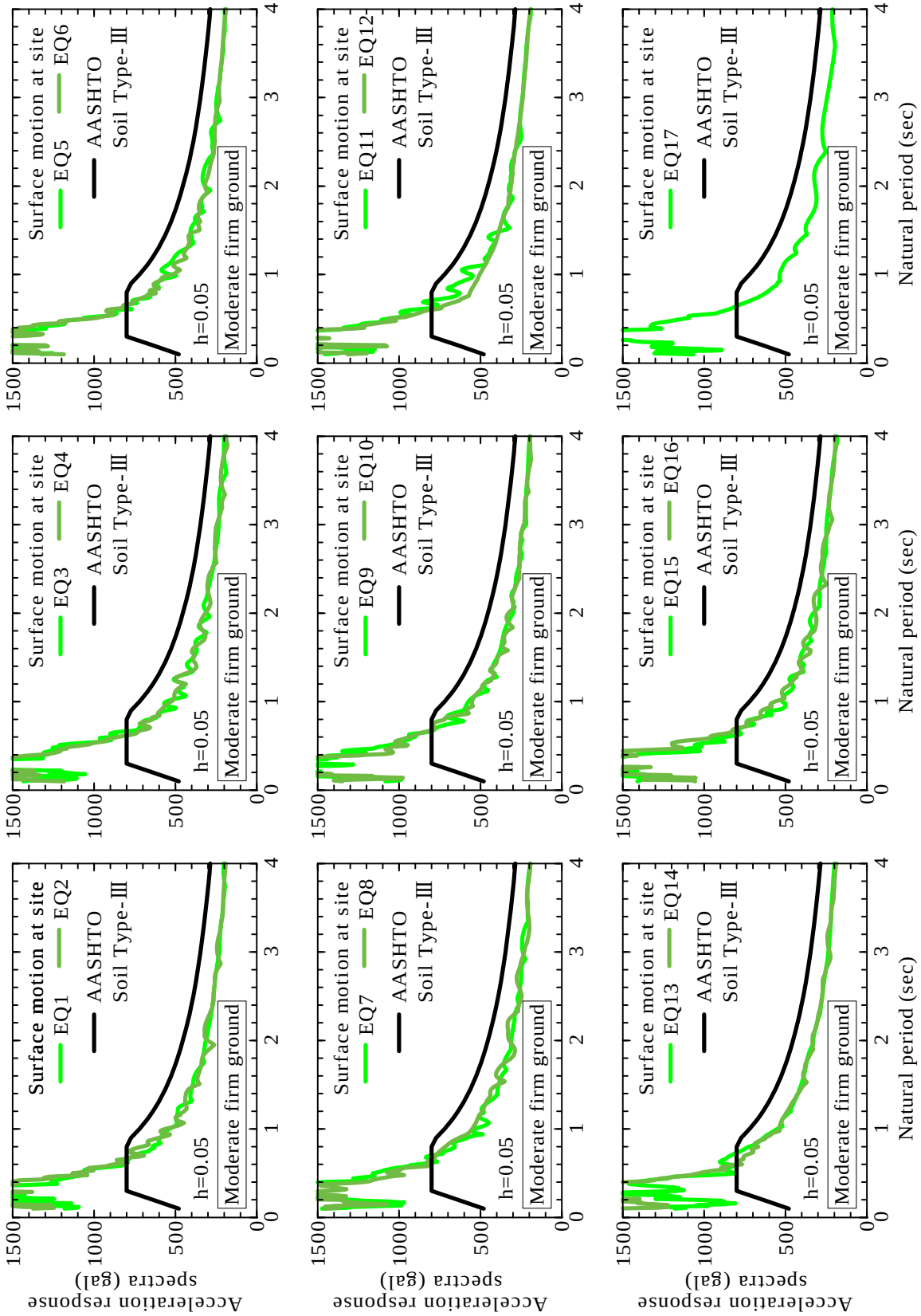


Figure 8.1.8-3 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate Firm Ground: Site No.1)

8.1.9 Development of Design Acceleration Response Spectra

(1) Propose of Design Acceleration Response Spectra

The shapes of ground surface acceleration response spectra at each point obtained from ground analysis have been evaluated, and shapes of design acceleration response spectra suitable for the ground conditions in the Philippines are proposed. In evaluating the acceleration response spectra at ground surface, the average of the acceleration response spectra obtained from all input earthquake ground motions was calculated in view of the variability of acceleration response spectra at ground surface depending on the phase of the input ground motion. In order to prevent inherent characteristics from being lost as a result of averaging, the maximum values of acceleration response spectra at different points were also evaluated. The concept of the method of determining acceleration response spectra at each point is shown below.

Mean of $S(T)$

$$S(T)_{mean} = \frac{\sum_{EQ=1}^{EQ=17} S(T)}{17} \quad \cdots\cdots (2)$$

Max of $S(T)$

$$S(T) = \max\left(S(T)_{EQ=1\sim17}\right) \quad \cdots\cdots (3)$$

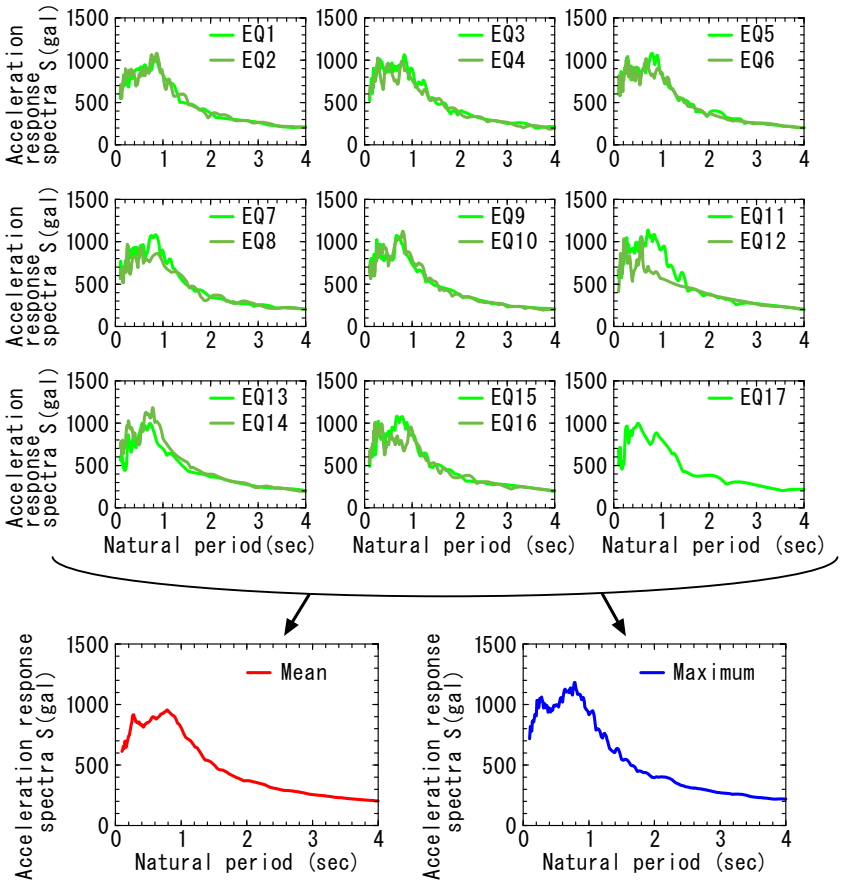


Figure 8.1.9-1 Estimation of Acceleration Spectra Response

Figure 8.1.9-2 shows the proposed shapes of design acceleration response spectra for the Philippines based on the acceleration response spectra at different points obtained from ground analysis. Table 8.1.9-1 to Table 8.1.9-3 show the proposed design formulas for design acceleration response spectra. Considerations in determining the acceleration response spectra to be proposed are as follows:

- The shapes design acceleration response spectra to be proposed have been determined so as to envelop the acceleration response spectra at ground surface obtained from the ground analysis results as much as possible.
- Since design acceleration response spectra vary considerably depending on earthquake ground motions, the average response (upper left in Figure 8.1.9-2) of the acceleration response spectra for 17 waves of ground motion was determined, and the maximum values of design acceleration response spectra to be proposed have been determined so that the average response distribution is enveloped as much as possible.
- The period characteristics to be proposed have been determined so as to envelop not only the average response distribution but also the maximum response distribution (lower left in Figure 8.1.9-2) as much as possible.
- The acceleration response spectra at Site 1, where ground is moderately firm, were not taken into consideration in determining design acceleration response spectra because they showed very large values compared with other sites (Site 2 to Site 6).
- The new formula for calculating design acceleration response spectra has been determined so that the AASHTO (2007) design acceleration response spectrum formula currently in use in the Philippines is not modified as much as possible.
- The use of another site coefficient has been proposed to take the effect of soil type into account. The conventional site coefficient (S) is used mainly to vary the range of periods showing the maximum values of acceleration response spectra according to soil type. The newly proposed site coefficient (S0) is used to vary the maximum values of acceleration response spectra according to soil type.

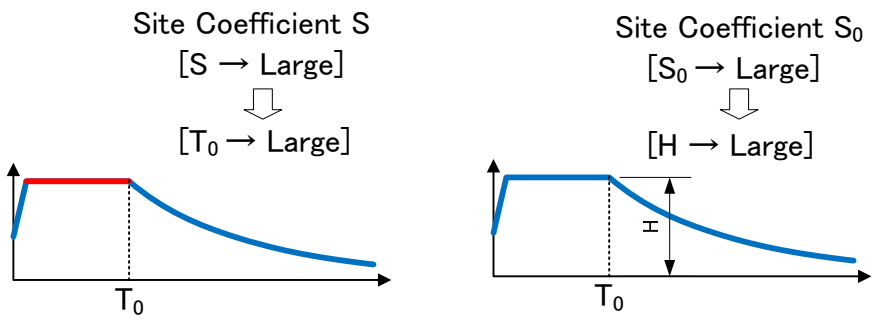


Figure 8.1.9-2 Roles of Site Coefficients S and S_0

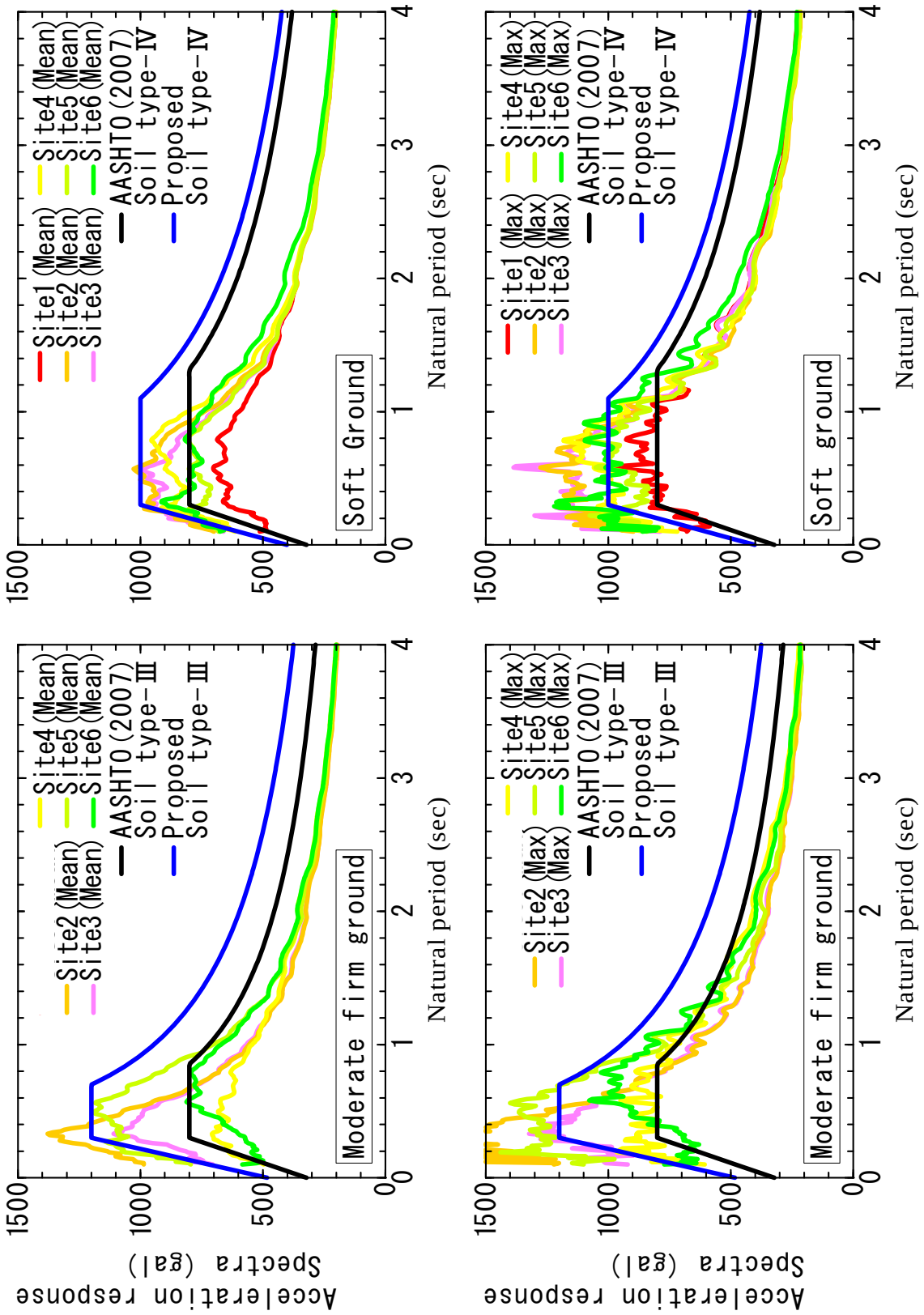


Figure 8.1.9-3 Proposed Design Acceleration Response Spectra Based on Study Results

Table 8.1.9-1 Proposed Acceleration Response Spectra Based on AASHTO (2007) (Moderate Firm Ground : Soil Type-III)

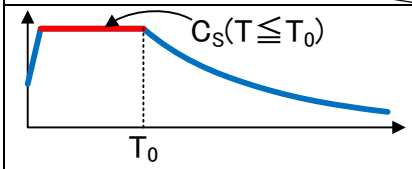
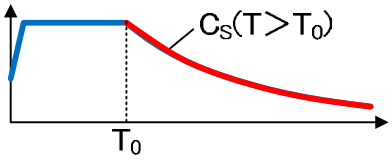
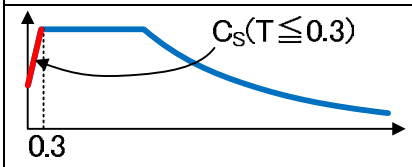
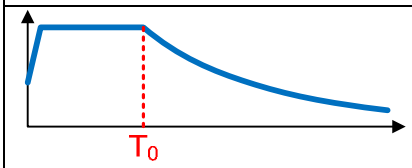
	AASHTO (2007) (= NSCP (2005))	Proposed
	$C_S = 2A$	$C_S = 2.5AS_0$
	$C_S = \frac{1.2AS}{T^{2/3}}$	$C_S = \frac{1.2AS \cdot S_0}{T^{2/3}}$
	$C_S = A(0.8 + 4.0T)$	$C_S = AS_0(1 + 5.0T)$ T=0, $C_S = AS_0$
	$T_0 = \left(\frac{1.2S}{2.0}\right)^{3/2}$	$T_0 = \left(\frac{1.2S}{2.5}\right)^{3/2}$
Site Coefficient	S=1.5	S_0=1.2, S=4.64

Table 8.1.9-2 Proposed acceleration response spectra based on AASHTO (2007) (Soft ground : Soil Type-IV)

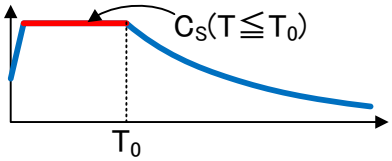
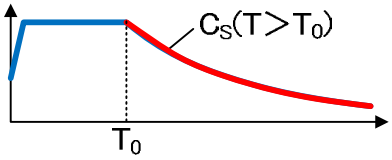
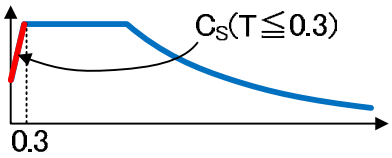
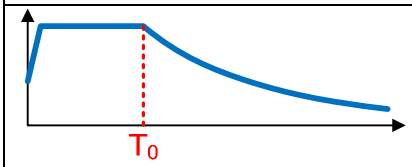
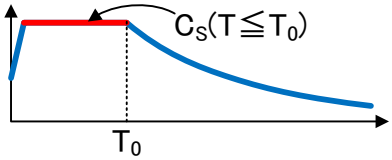
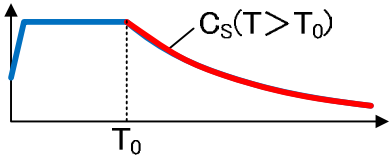
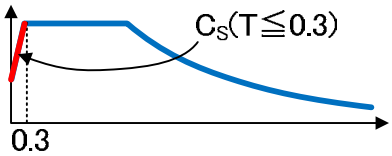
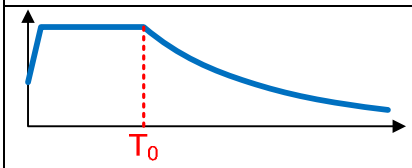
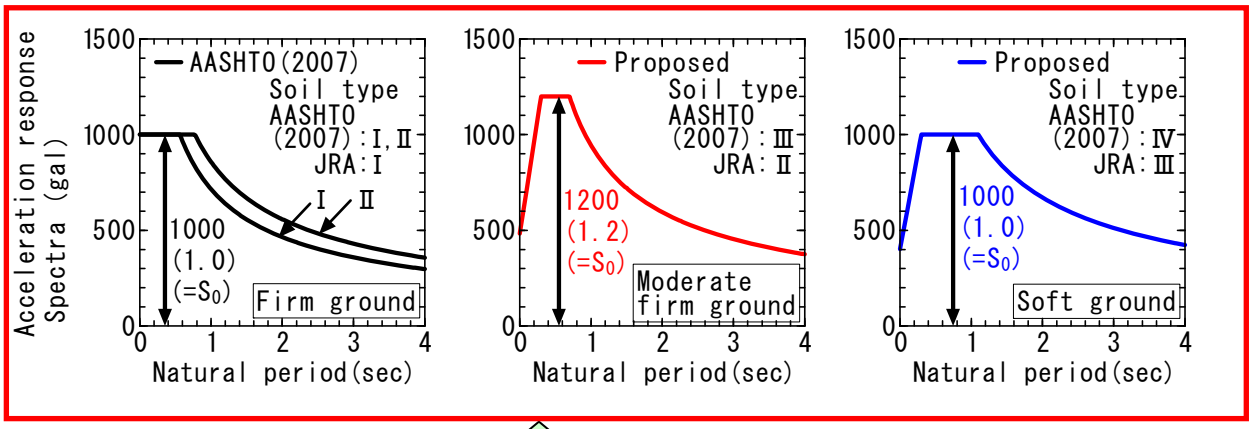
	AASHTO (2007)	Proposed
	$C_S = 2A$	$C_S = 2.5AS_0$
	$C_S = \frac{1.2AS}{T^{2/3}}$	$C_S = \frac{1.2AS \cdot S_0}{T^{2/3}}$
	$C_S = A(0.8 + 4.0T)$	$C_S = AS_0(1 + 5.0T)$ T=0, $C_S = AS_0$
	$T_0 = \left(\frac{1.2S}{2.0}\right)^{3/2}$	$T_0 = \left(\frac{1.2S}{2.5}\right)^{3/2}$
Site Coefficient	S=2.0	S_0=1.0, S=2.22

Table 8.1.9-3 Proposed Acceleration Response Spectra Based on AASHTO (2007) (Soil Type –I, II, III, IV)

	Proposed
	$C_S = 2.5AS_0$
	$C_S = \dfrac{1.2AS \cdot S_0}{T^{2/3}}$
	$C_S = AS_0(1+5.0T)$ T=0, $C_S = AS_0$
	$T_0 = \left(\dfrac{1.2S}{2.5}\right)^{3/2}$
Site Coefficient (S_0)	(I, II, III, IV)=(1.0, 1.0, 1.2, 1.0)
Site Coefficient (S)	(I, II, III, IV)=(1.0, 1.2, 1.64, 2.22)

(2) Comparison Proposed Spectra and Design Spectra of AASHTO (2012)

The ratios of the values for moderately firm ground and soft ground relative to the values for firm ground (= 1.0) are close to the ratios in AASHTO (2012).



AASHTO 2012	Proposed S_0
1.0	1.0
1.1	1.2
0.9	1.0

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (SS)					Soil type
	SS< 0.25	SS= 0.50	SS= 0.75	SS= 1.00	SS> 1.25	
A	0.8	0.8	0.8	0.8	0.8	
B	1.0	1.0	1.0	1.0	1.0	
C	1.2	1.2	1.1	1.0	1.0	Firm
D	1.6	1.4	1.2	1.1	1.0	Moderate Firm ground
E	2.5	1.7	1.2	0.9	0.9	Soft ground
F	-	-	-	-	-	

Figure 8.1.9-4 Comparison Proposed Spectra and Design Spectra of AASHTO (2012)

8.1.10 Conclusion

- Ground response analyses were conducted, and ground surface acceleration response spectra were calculated, taking the ground characteristics of the Philippines into consideration.
- Comparison of the ground surface acceleration response spectra obtained from the ground response analysis and the AASHTO (2007) design acceleration response spectra has confirmed that there are some differences in maximum values and period characteristics.
- On the basis of the comparison results mentioned above, shapes of design acceleration response spectra based on the AASHTO (2007) design acceleration response spectra appropriate for the ground characteristics of the Philippines have been proposed.

8.2 Method 2 – Based on Probabilistic Seismic Hazard Analysis

Site-specific design spectra are obtained for 7 bridge sites (2 bridges in Package B and 5 bridges in
Package C). The results shall be used in the outline design of the selected bridges.

Site-specific design spectra at a location are obtained using the procedure shown in Figure 8.2-1 and Figure 8.2-2. It basically consists of conducting a probabilistic seismic hazard analysis and a dynamic site response analysis.

Active faults as presently identified by PHIVOLCS are shown plotted in Figure 8.2-3.
Earthquake events from 1907 to 2012 (plotted in Figure 8.2-3) that were instrumentally recorded are compiled in the earthquake catalog. The magnitude scale is homogenized in a common moment magnitude scale. Declustering algorithm is applied to retain only independent main shocks (as shown plotted in Figure 8.2-4), removing aftershocks and foreshocks. Seismic source modeling consisting of fault models and background seismicity models are shown in Figure 8.2-4.

All significant seismic sources within a radius of 300 kilometers that could potentially contribute to significant ground shaking to the site of interest are usually included in the PSHA computation.

Uniform hazard spectral curves for the basement rock at Guadalupe Bridge (see Figure 8.2-5) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-6. Site-specific design spectra are generated for four locations at the Guadalupe Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level (assumed at 30 to 45 meters deep) and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-7 (for A1), Figure 8.2-8 (for C2), Figure 8.2-9 (for B2), and Figure 8.2-10 (for D2). Further, site-specific design spectra at the 4 locations are compared at each return period, as shown in Figure 8.2-11 (for 50-year return period), Figure 8.2-12 (for 100-year return period), Figure 8.2-13 (for 500-year return period), and Figure 8.2-14 (for 1000-year return period).

Uniform hazard spectral curves for the basement rock at Lambingan Bridge (see Figure 8.2-15 and Figure 8.2-16) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-17. Site-specific design spectra are generated for two locations at the Lambingan Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-18 (for A1), and Figure 8.2-19 (for B2).

Uniform hazard spectral curves for the basement rock at Palanit Bridge (see Figure 8.2-20) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-21. Site-specific design spectra are generated for location A1 at the Palanit Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-22.

Uniform hazard spectral curves for the basement rock at Mawo Bridge (see Figure 8.2-23) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-24. Site-specific design spectra are generated for two locations at the Mawo Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-25 (for A1), and Figure 8.2-26 (for B2).

Uniform hazard spectral curves for the basement rock at Liloan Bridge (see Figure 8.2-27) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-28. Site-specific design spectra are generated for A1 at Liloan Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-29.

Uniform hazard spectral curves for the basement rock at 1st Mactan-Mandaue Bridge (see Figure 8.2-30) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-31. Site-specific design spectra are generated for two locations at the 1st Mactan-Mandaue Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-32 (for A1), and Figure 8.2-33 (for B2).

Last but not the least, uniform hazard spectral curves for the basement rock at Wawa Bridge (see Figure 8.2-34) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 8.2-35. Site-specific design spectra are generated for A1 at Wawa Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 8.2-36.

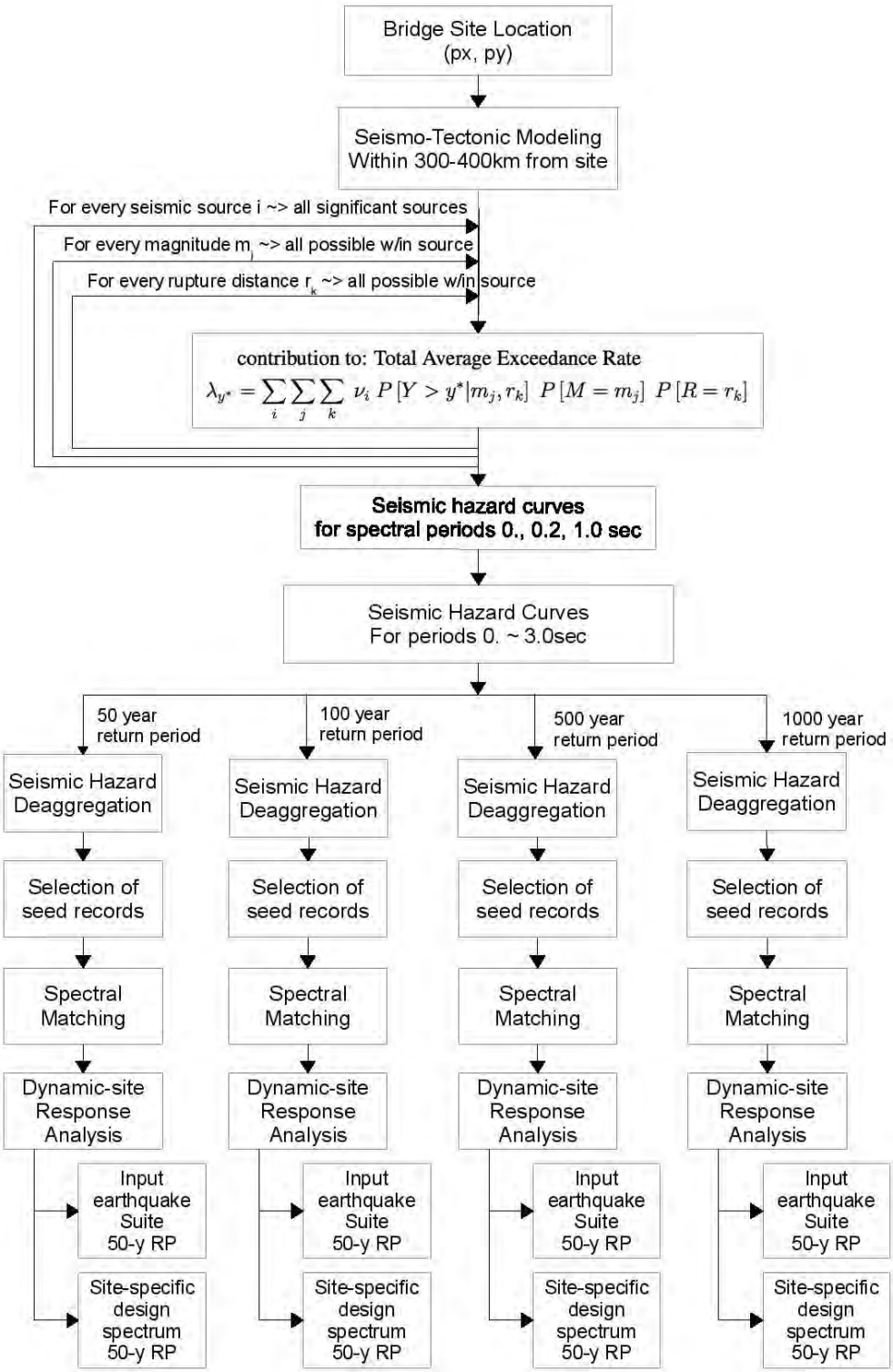


Figure 8.2-1 Procedure of PSHA study for site-specific design spectra corresponding to return periods of 50, 100, 500, and 1000 years

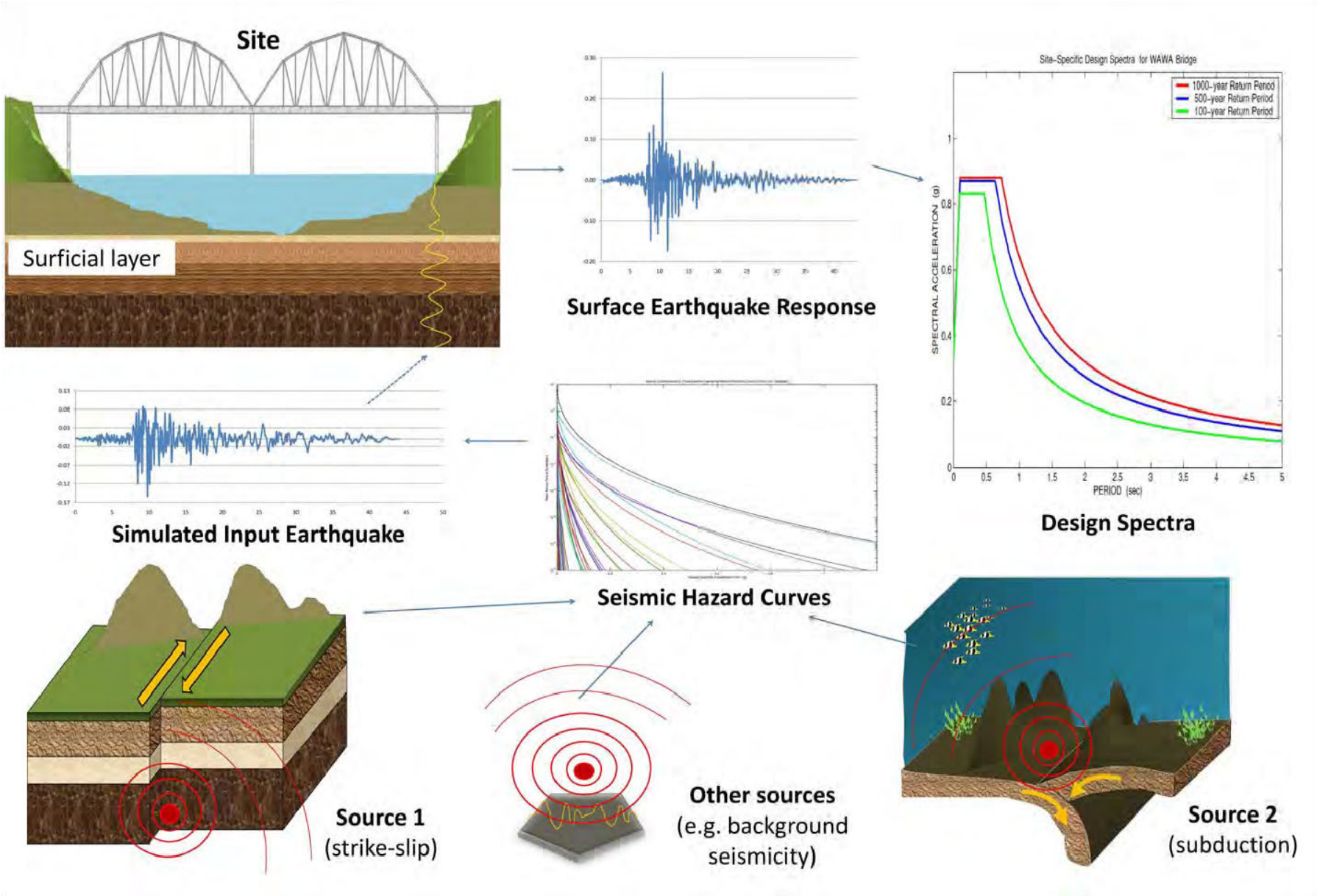


Figure 8.2-2 Schematic analysis flow for generating site-specific design spectrum

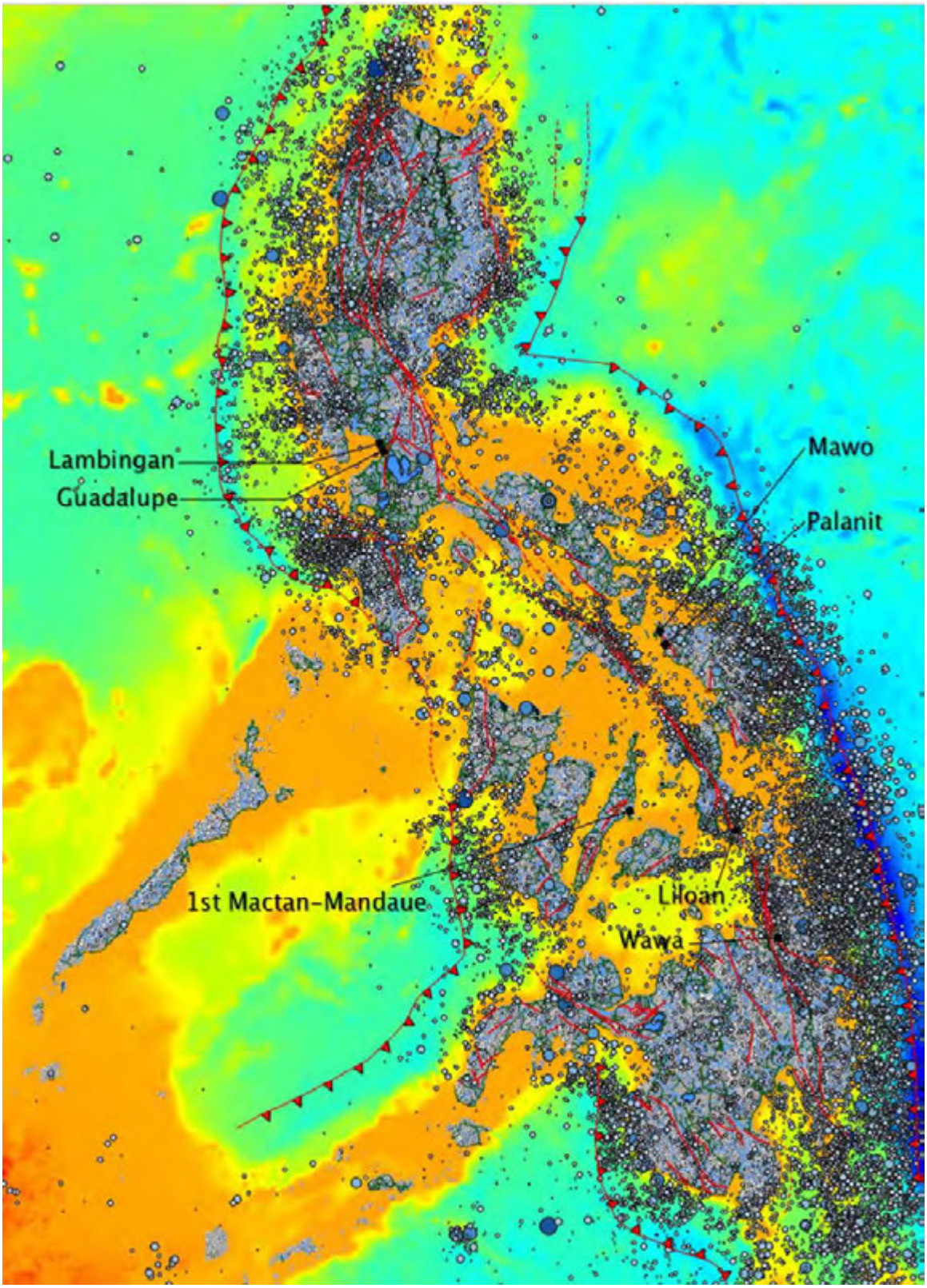


Figure 8.2-3 Philippine seismological and tectonic setting (earthquakes 1907–2012 with depth < 100 kms;fault traces after PHIVOLCS)

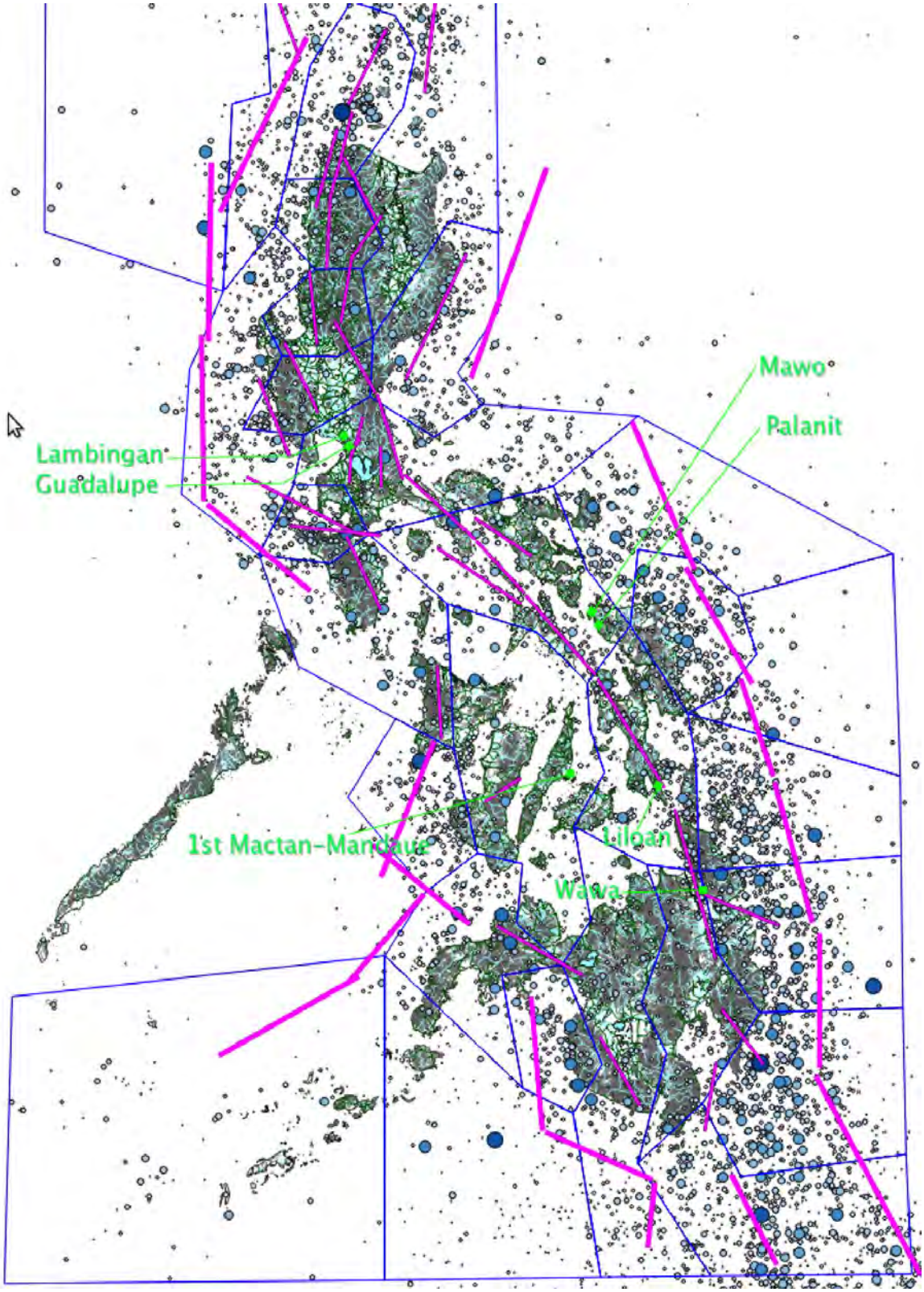
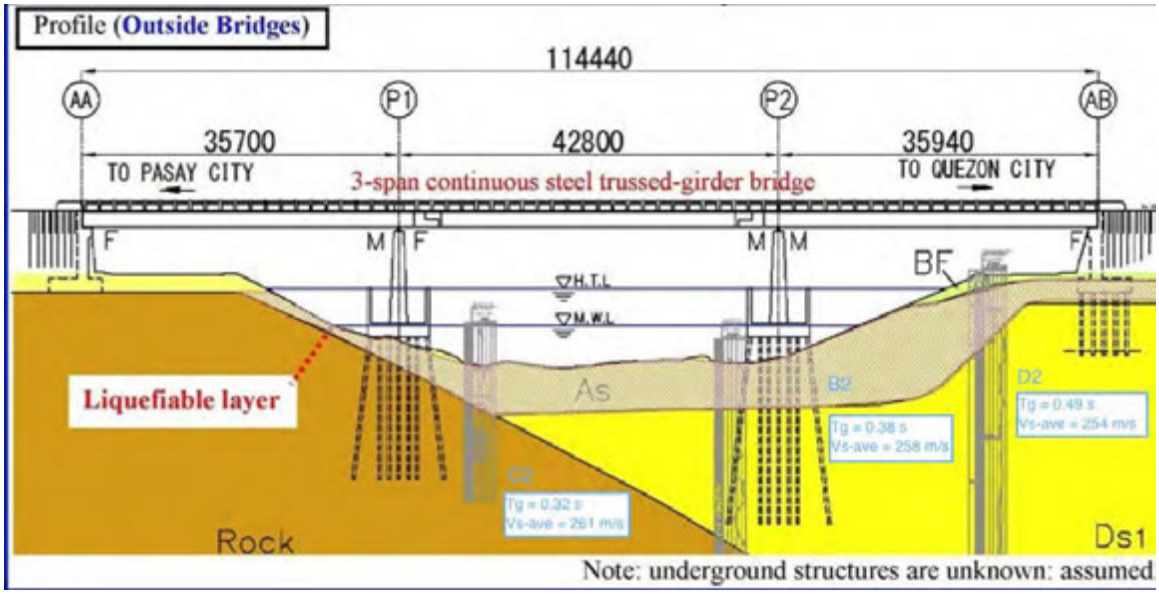


Figure 8.2-4 Seismic source modeling used in this study
— fault models and background seismicity models
(also plotted are declustered earthquakes from 1907 to 2012)

(a) site plan



(b) site profile

Figure 8.2-5 Guadalupe Bridge site location and data

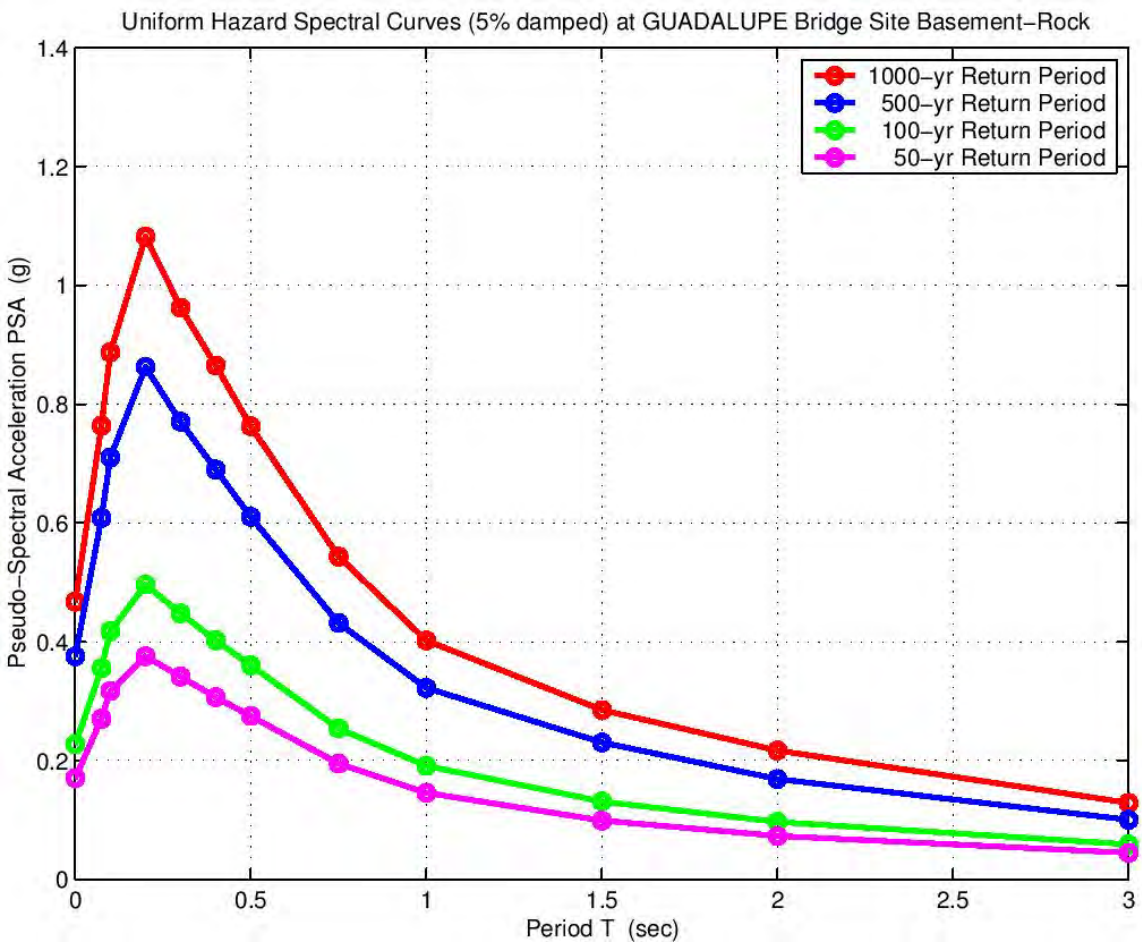


Figure 8.2-6 Uniform hazard spectral curve for Guadalupe Bridge site at basement-rock

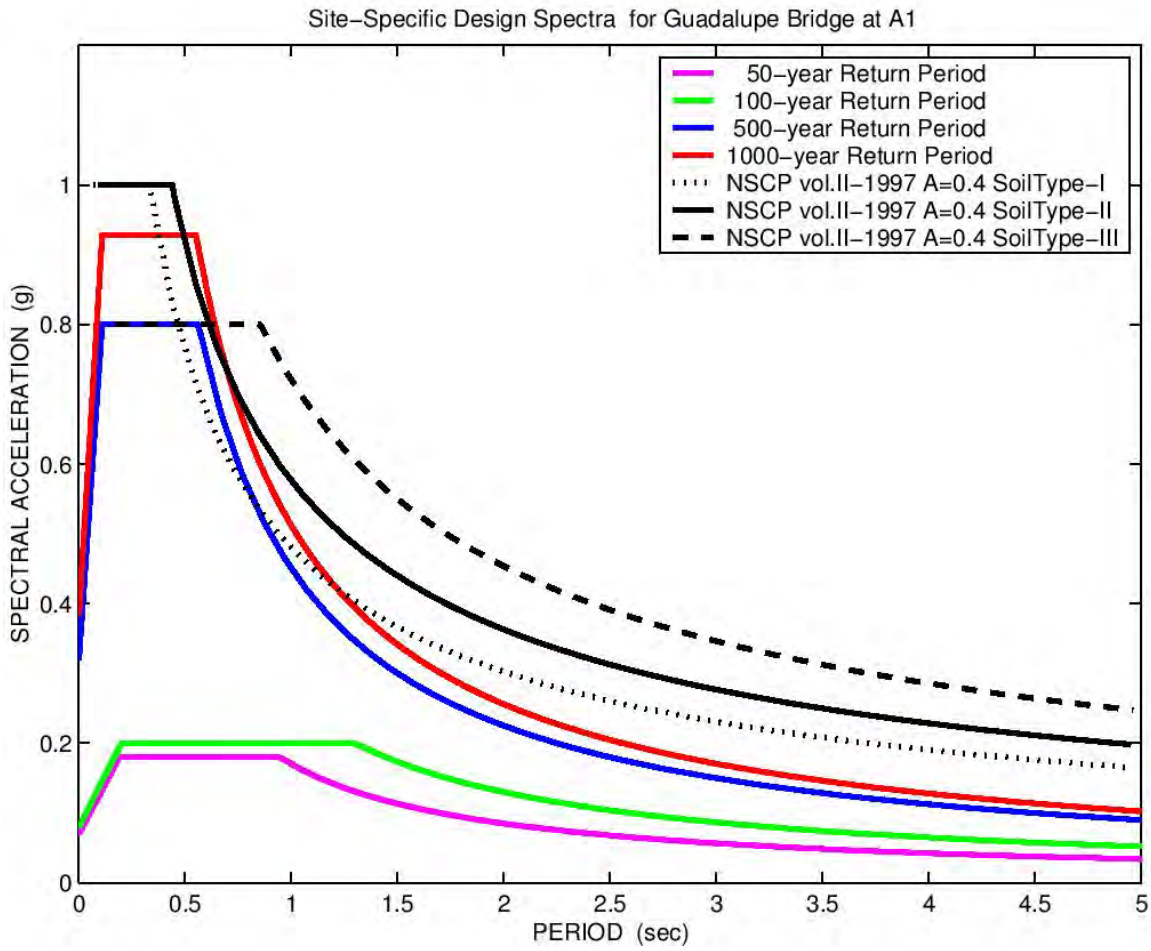


Figure 8.2-7 Site-specific design spectra for Guadalupe Bridge at A1

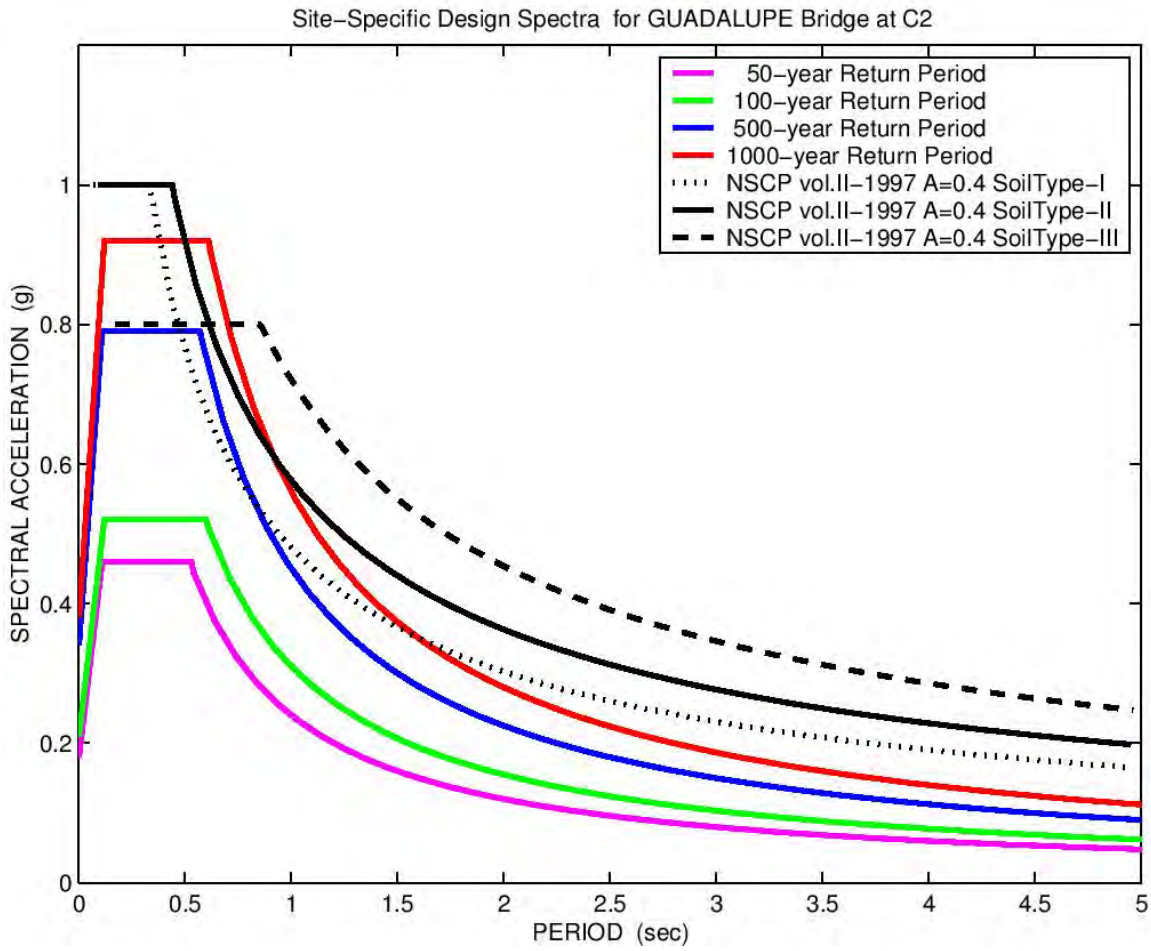


Figure 8.2-8 Site-specific design spectra for Guadalupe Bridge at C2

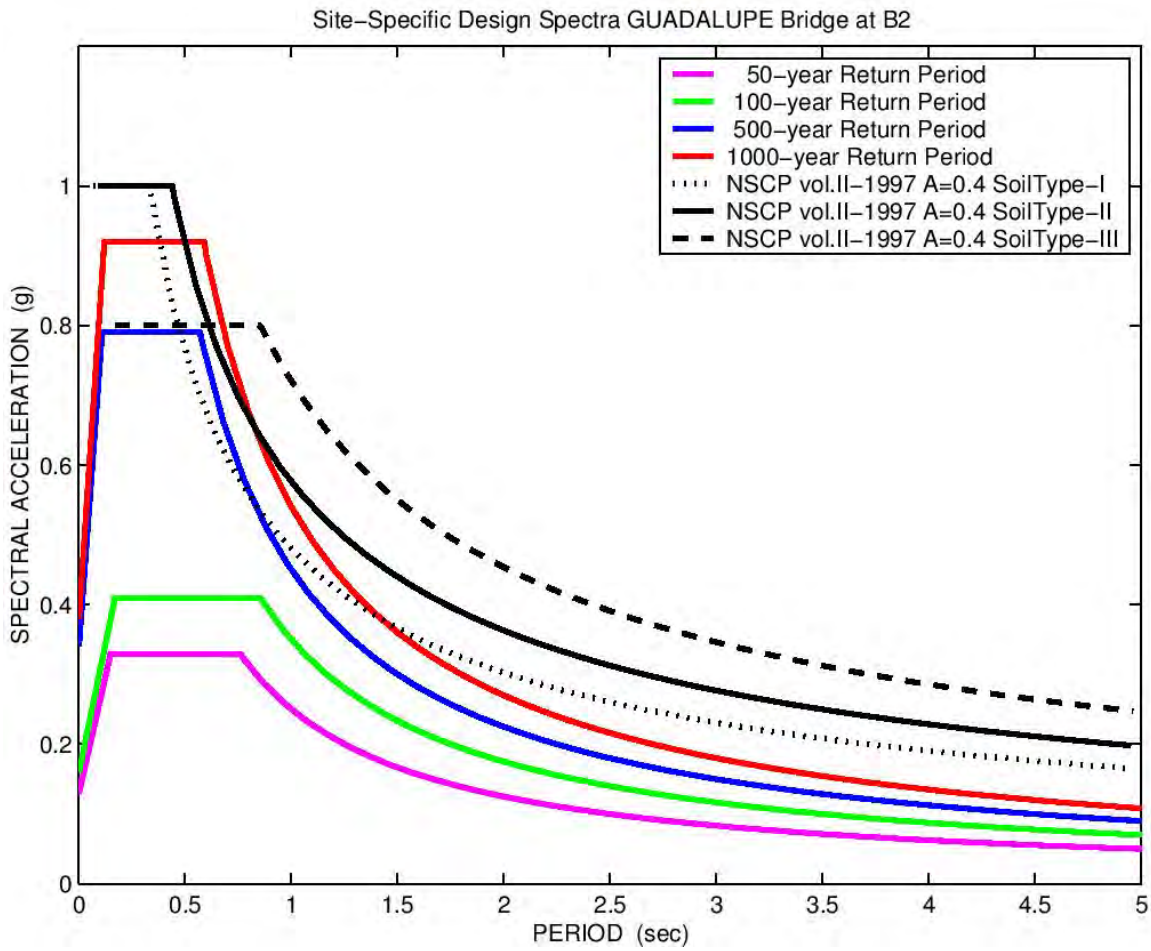


Figure 8.2-9 Site-specific design spectra for Guadalupe Bridge at B2

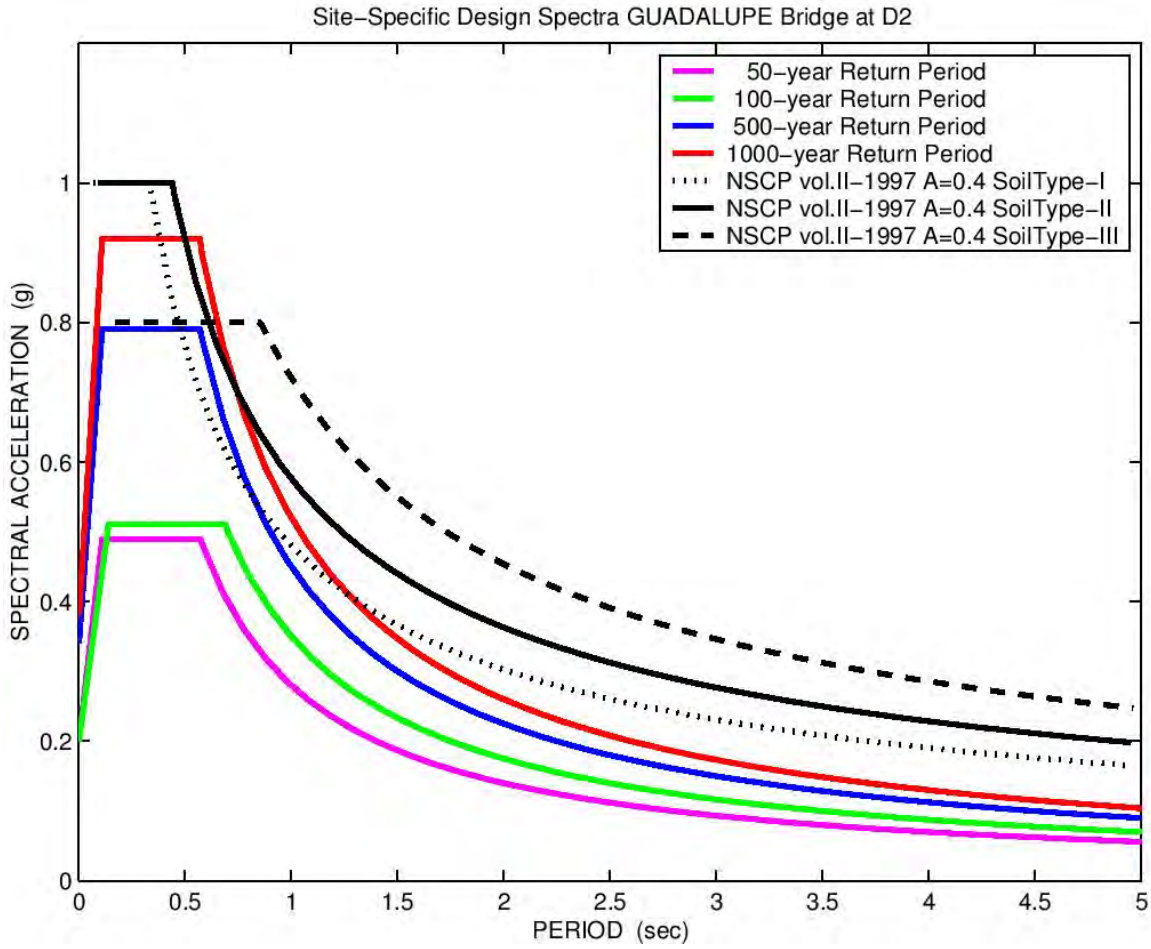


Figure 8.2-10 Site-specific design spectra for Guadalupe Bridge at D2

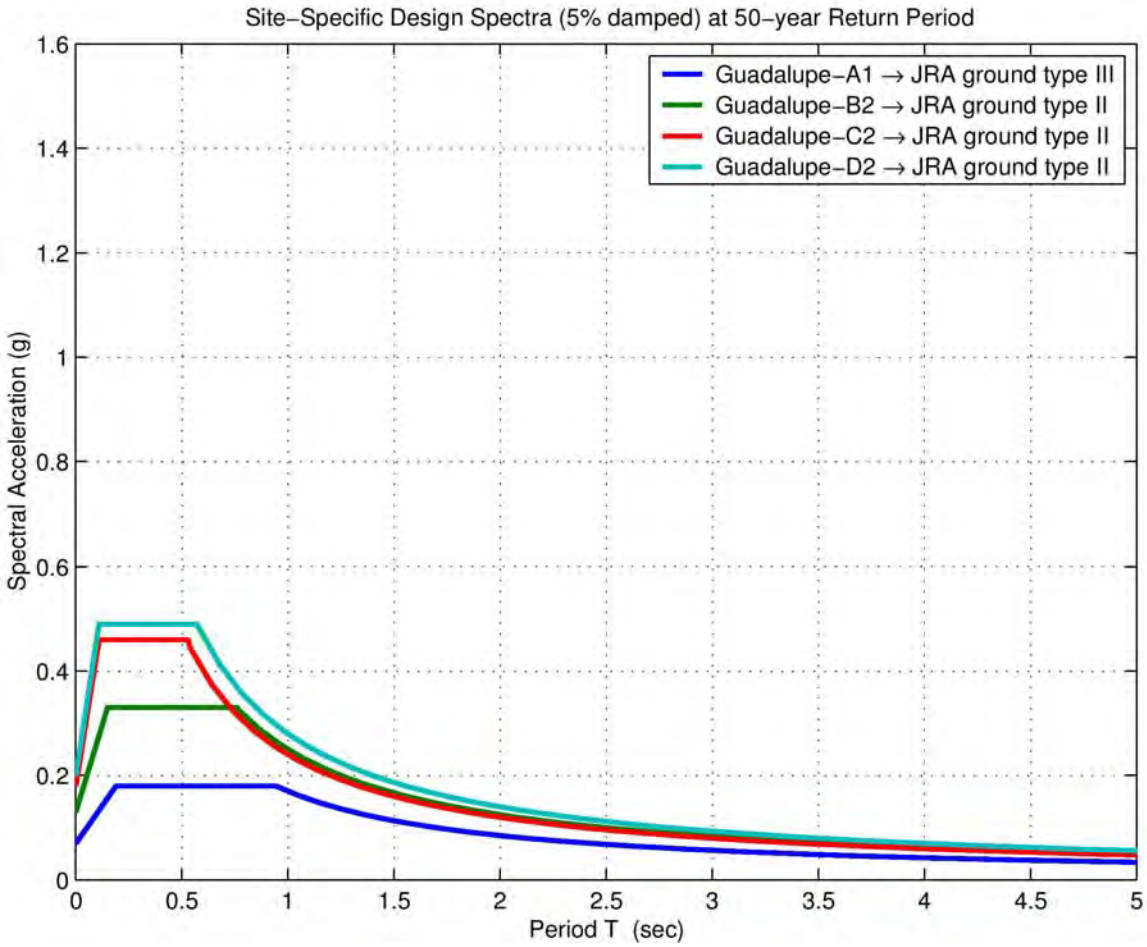


Figure 8.2-11 Site-specific design spectra (50-year return period) for Guadalupe Bridge

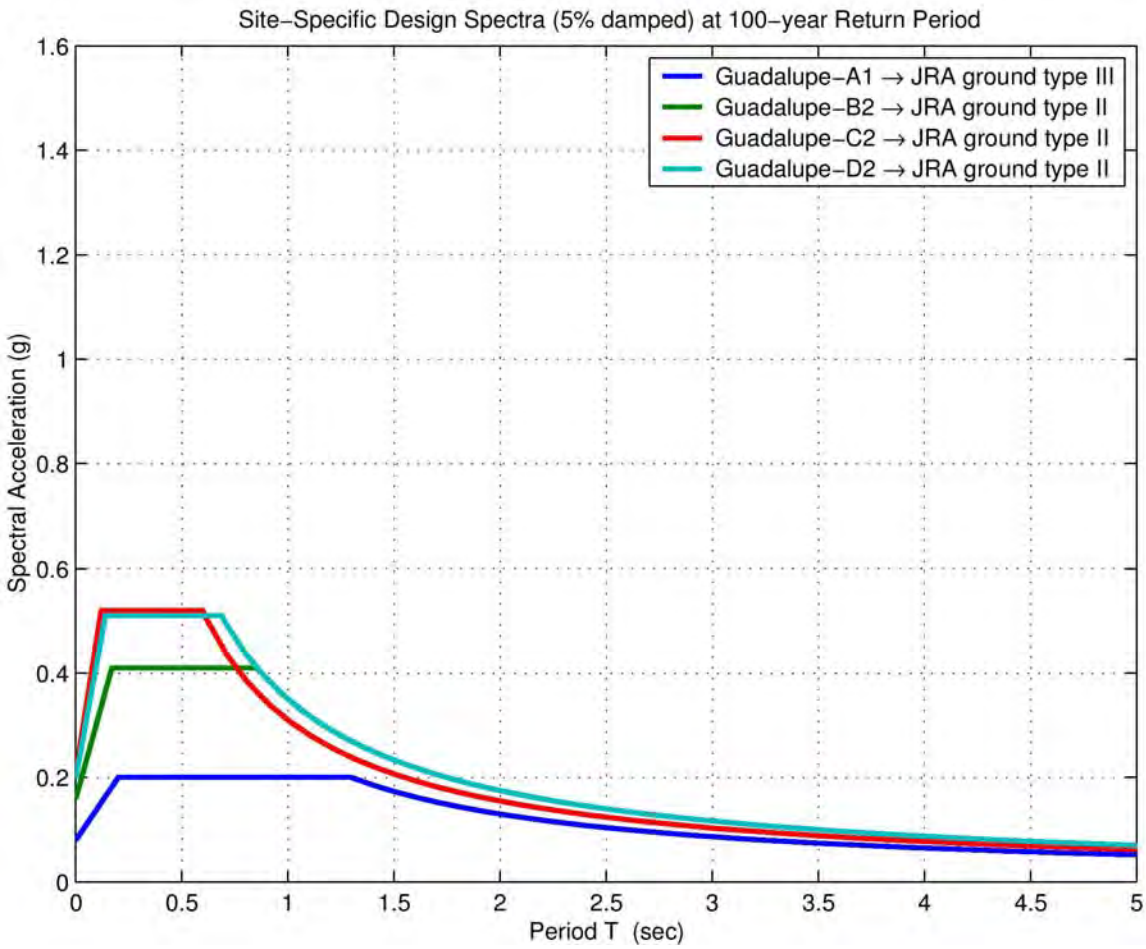


Figure 8.2-12 Site-specific design spectra (100-year return period) for Guadalupe Bridge

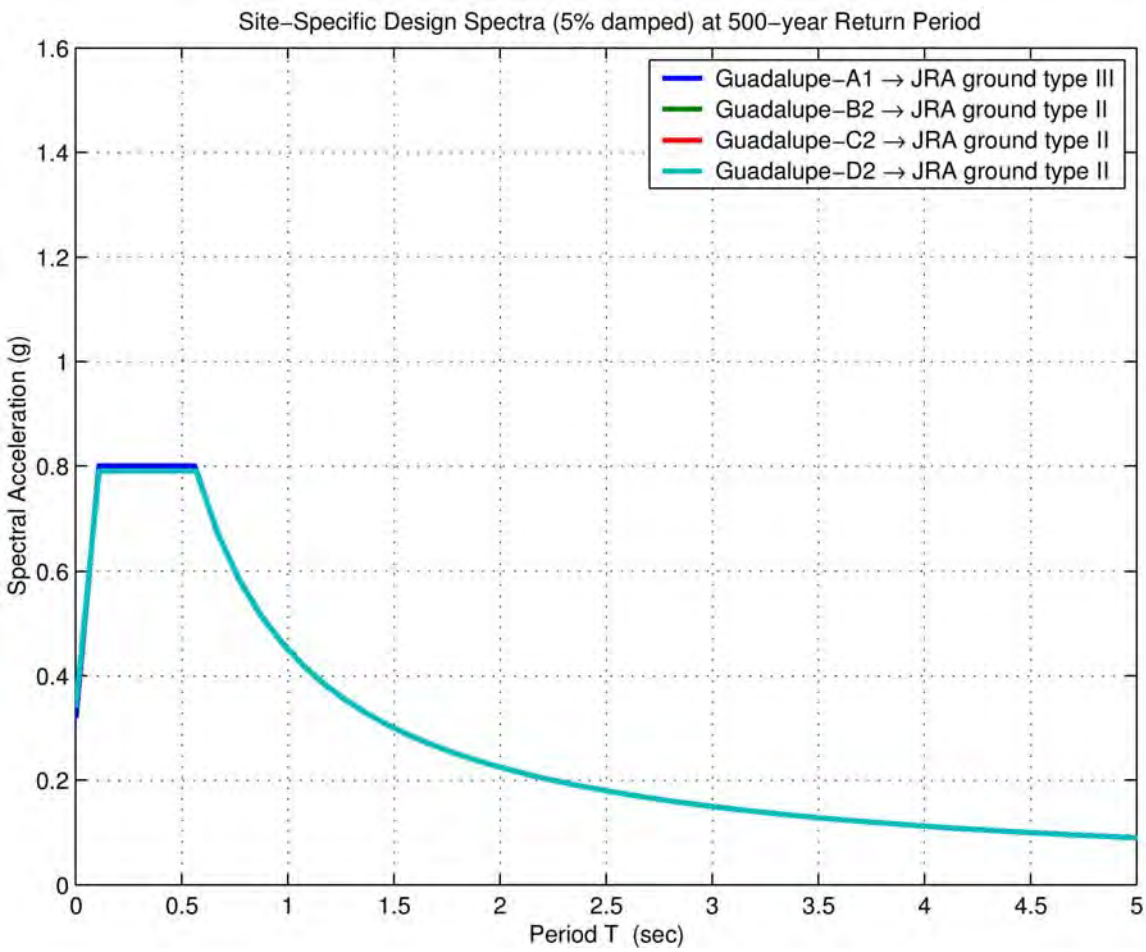


Figure 8.2-13 Site-specific design spectra (500-year return period) for Guadalupe Bridge

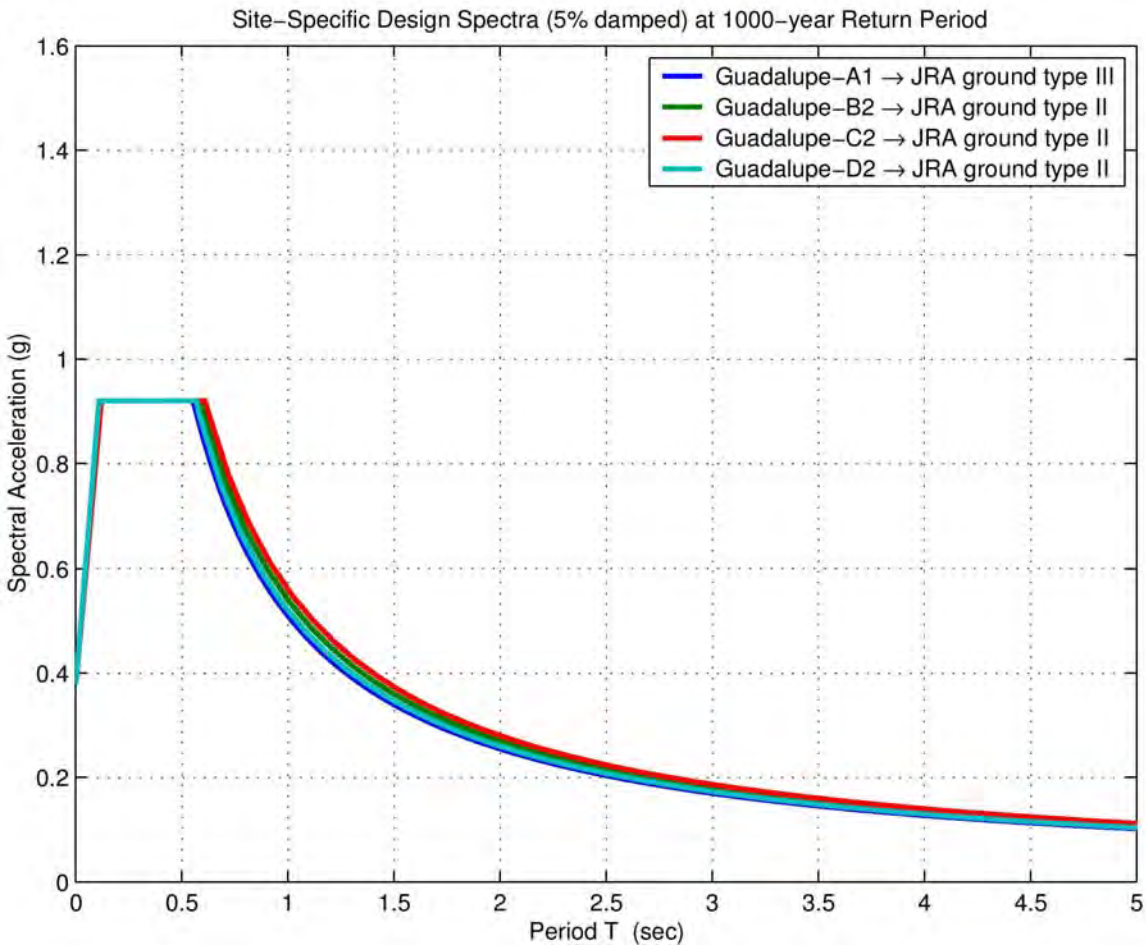
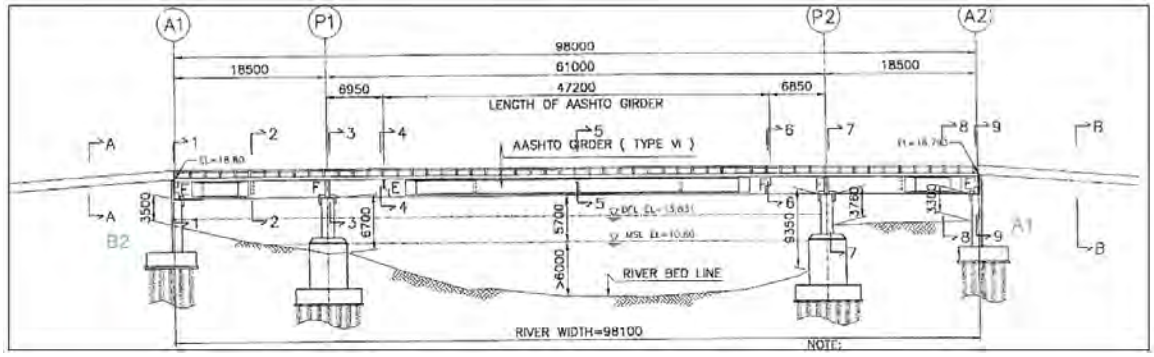


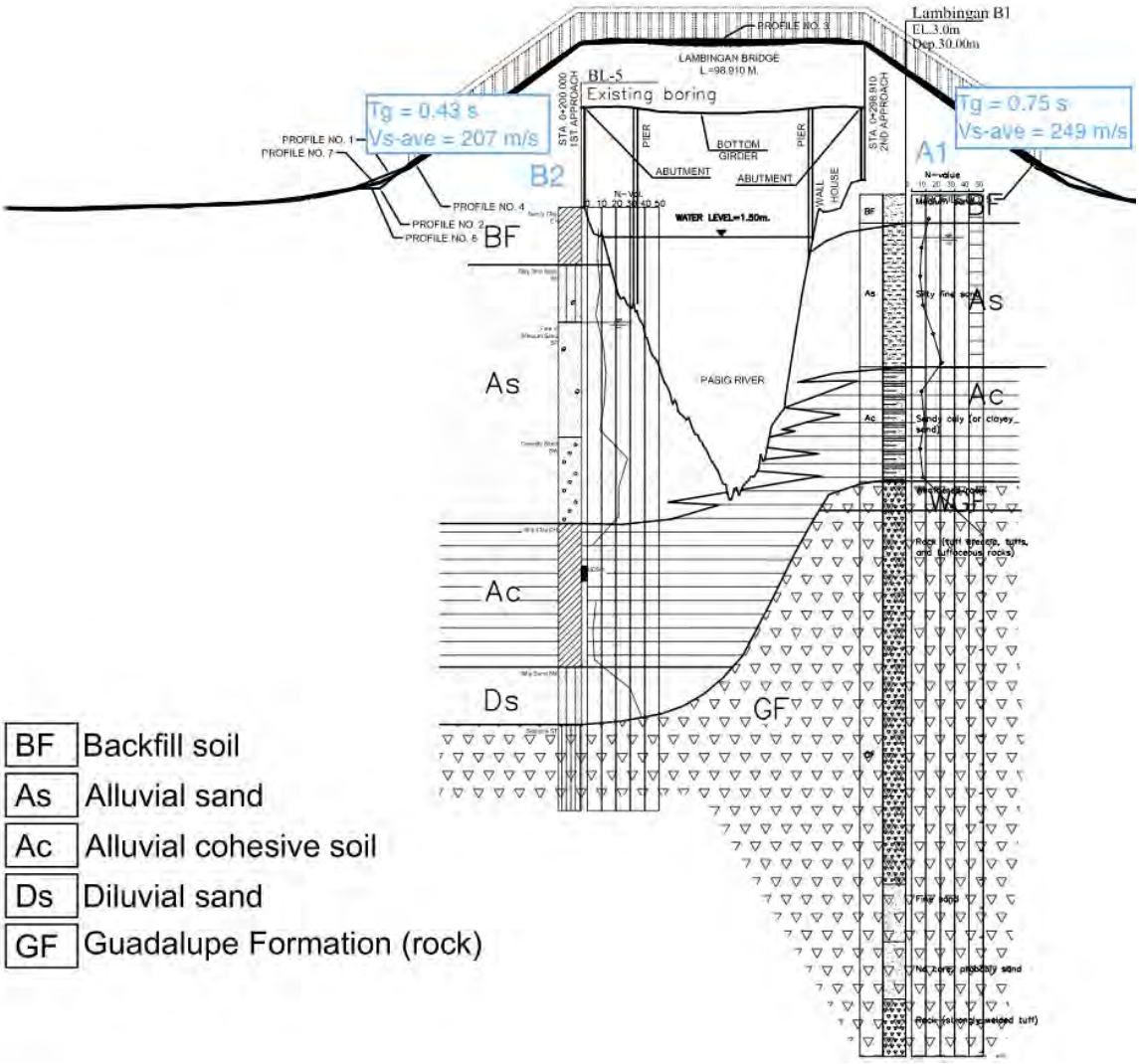
Figure 8.2-14 Site-specific design spectra (1000-year return period) for Guadalupe Bridge

(a) site plan



(b) bridge profile

Figure 8.2-15 Lambingan Bridge site location and data



(c) soil profile

Figure 8.2-16 Lambingan Bridge site location and data

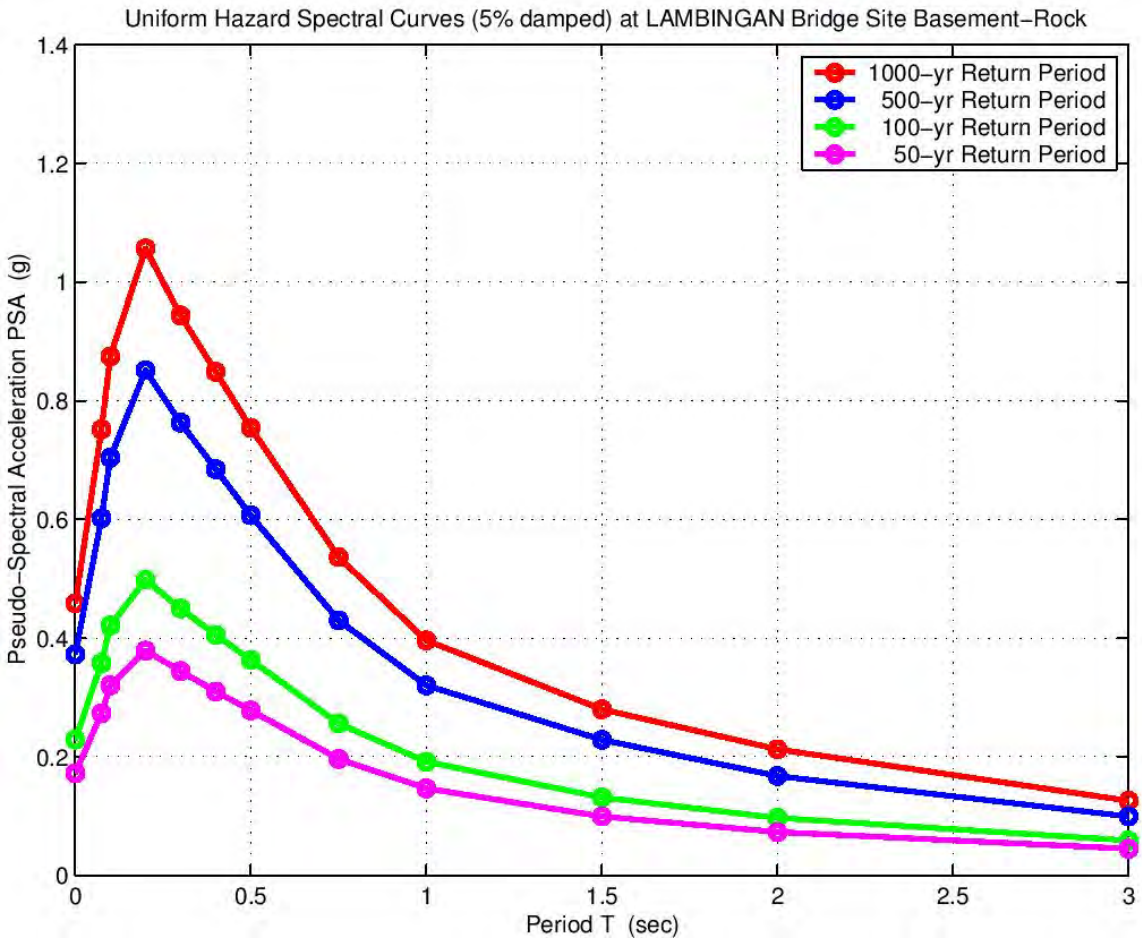


Figure 8.2-17 Uniform hazard spectral curve for Lambingan Bridge site at basement-rock

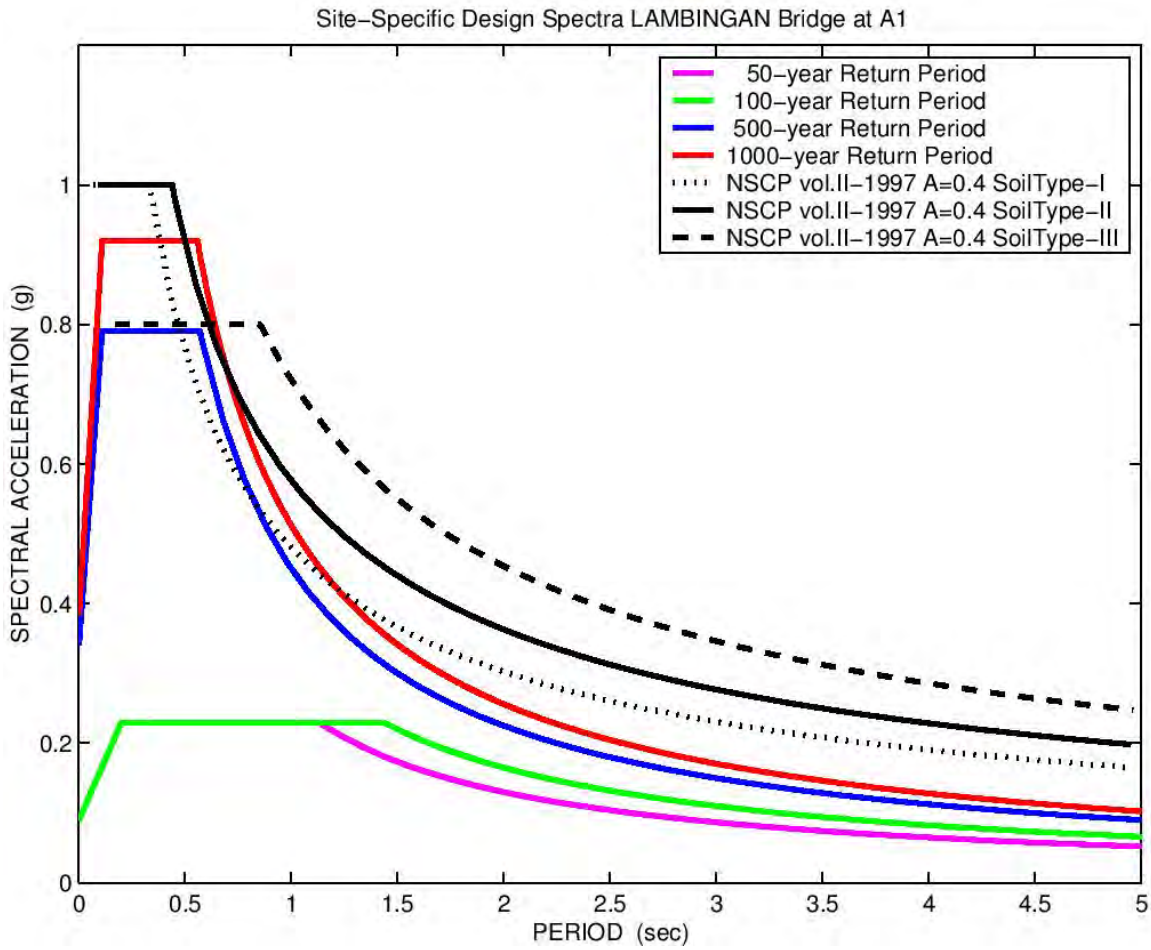


Figure 8.2-18 Site-specific design spectra for Lambingan Bridge at A1

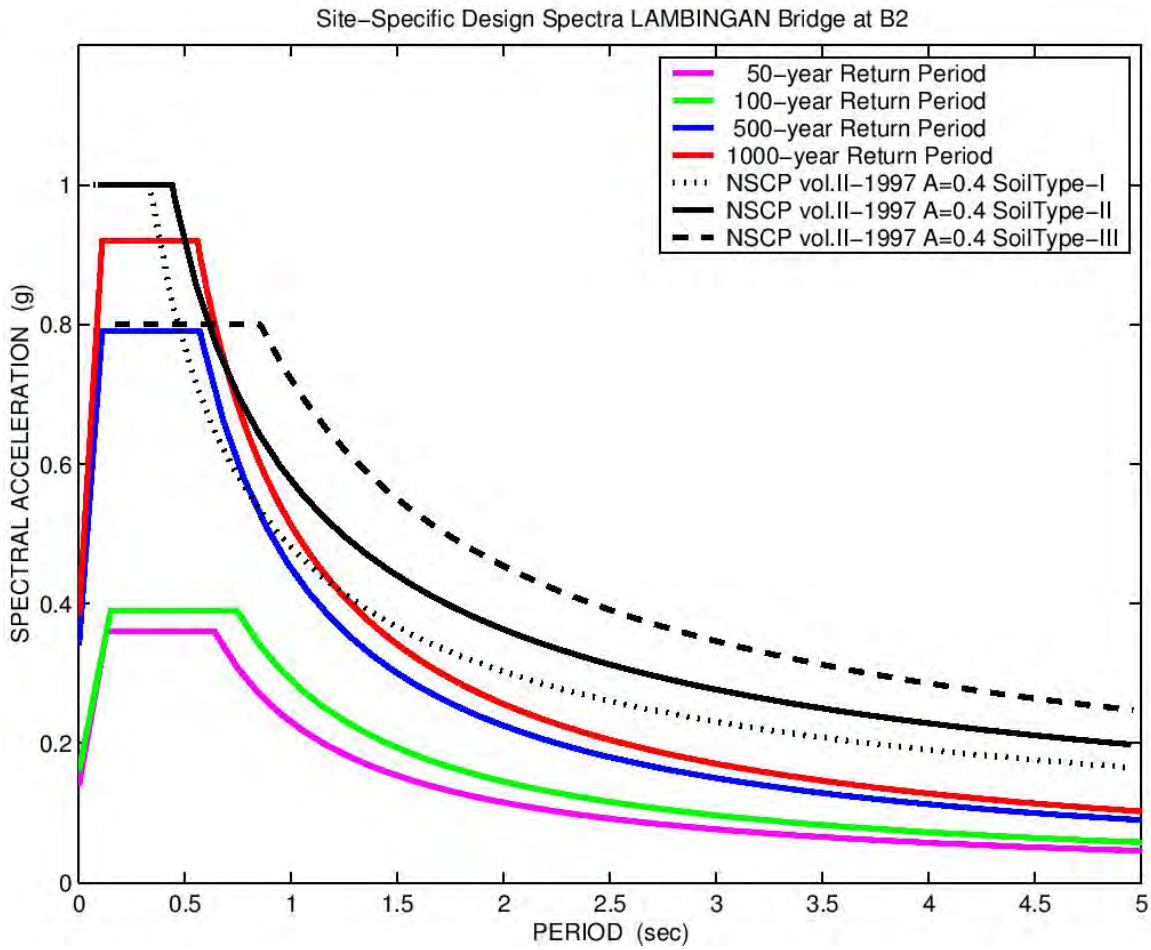


Figure 8.2-19 Site-specific design spectra for Lambingan Bridge at B2

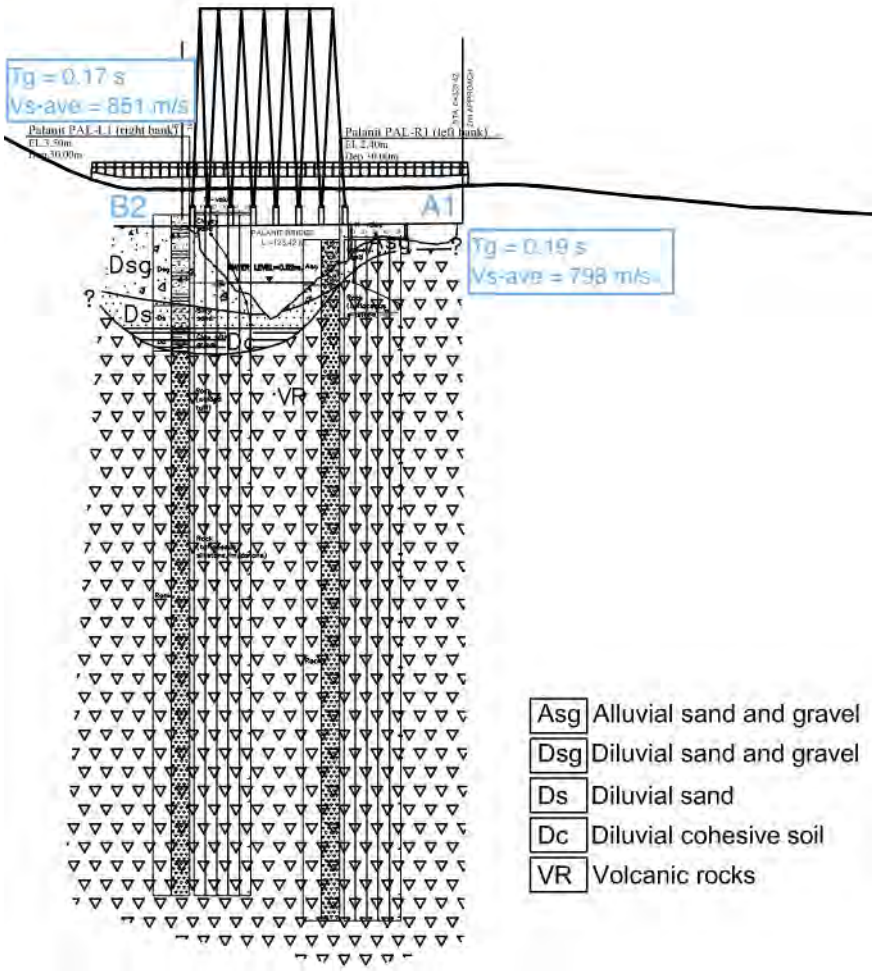


Figure 8.2-20 Palanit Bridge site data

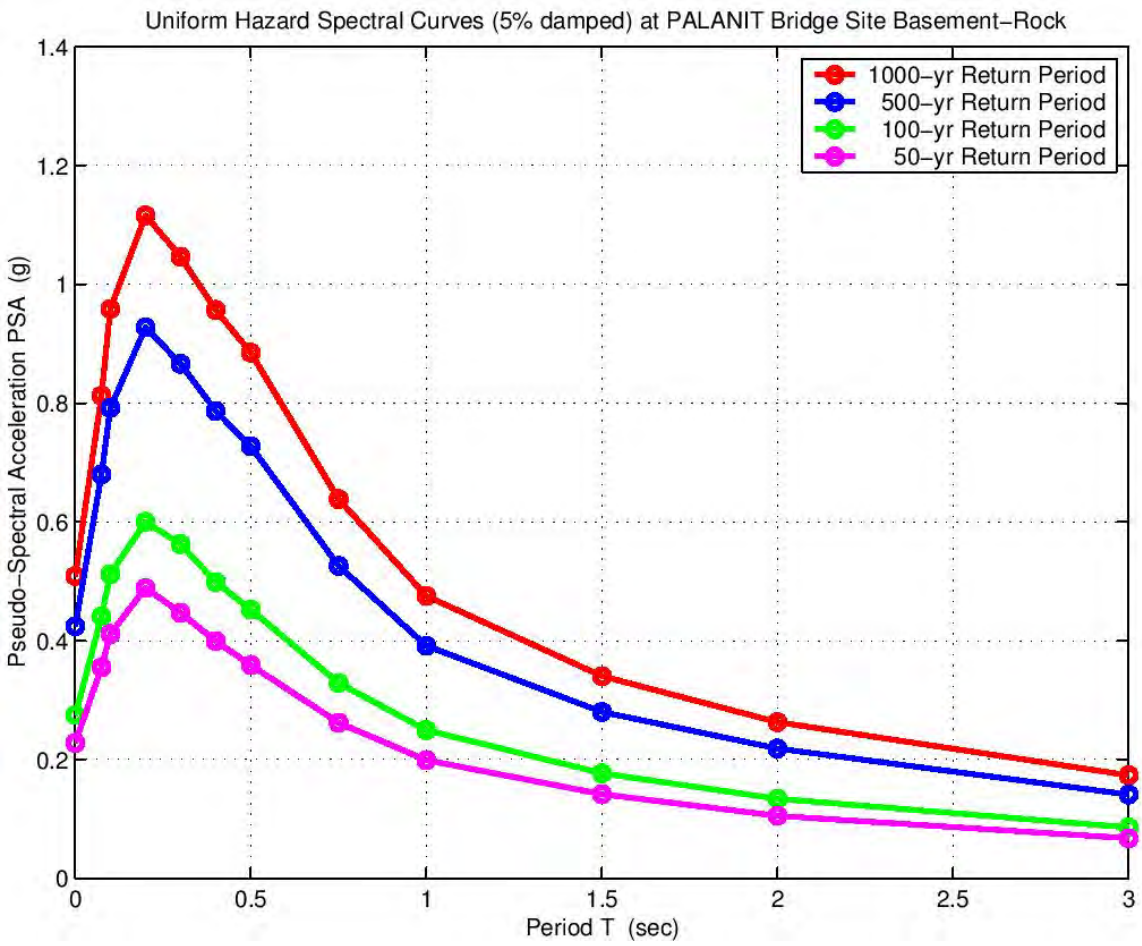


Figure 8.2-21 Uniform hazard spectral curve for Palanit Bridge site at basement-rock

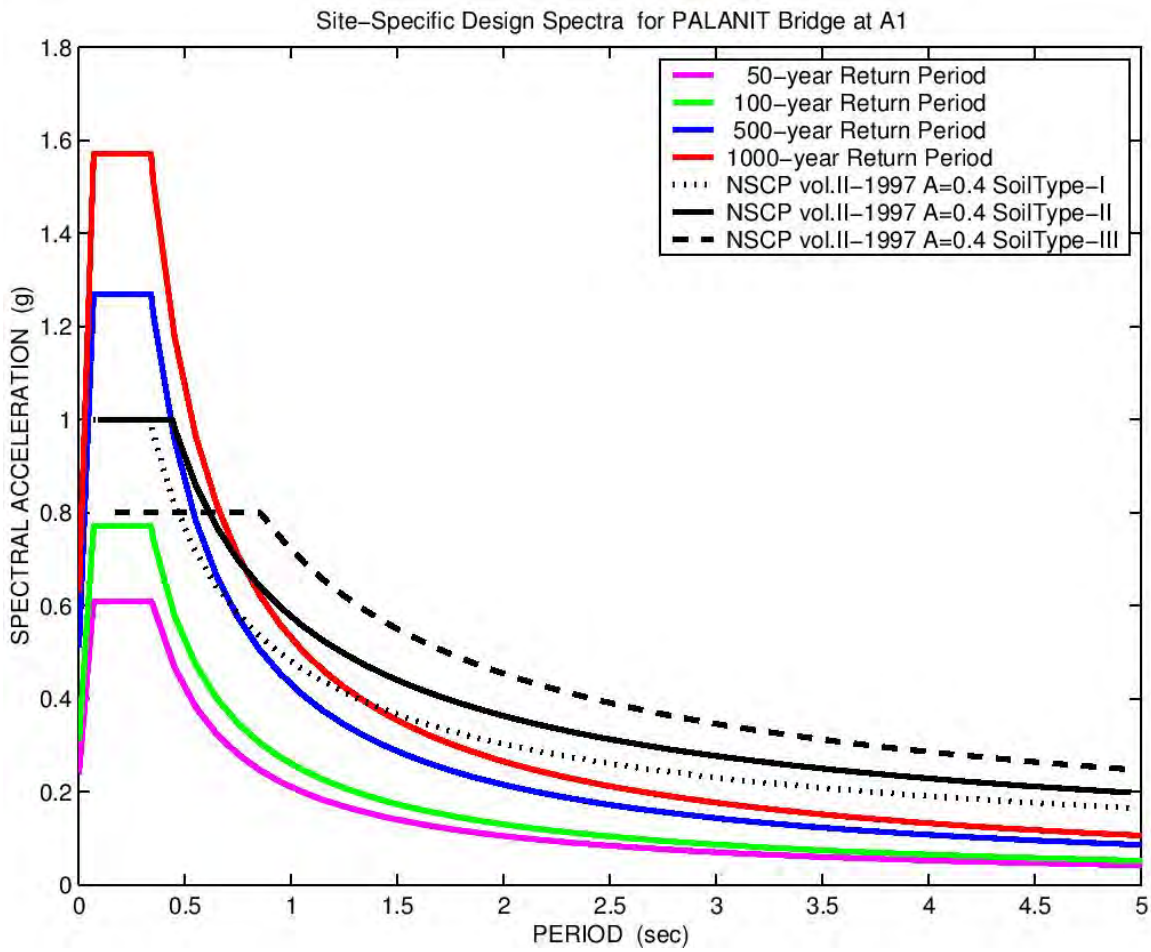
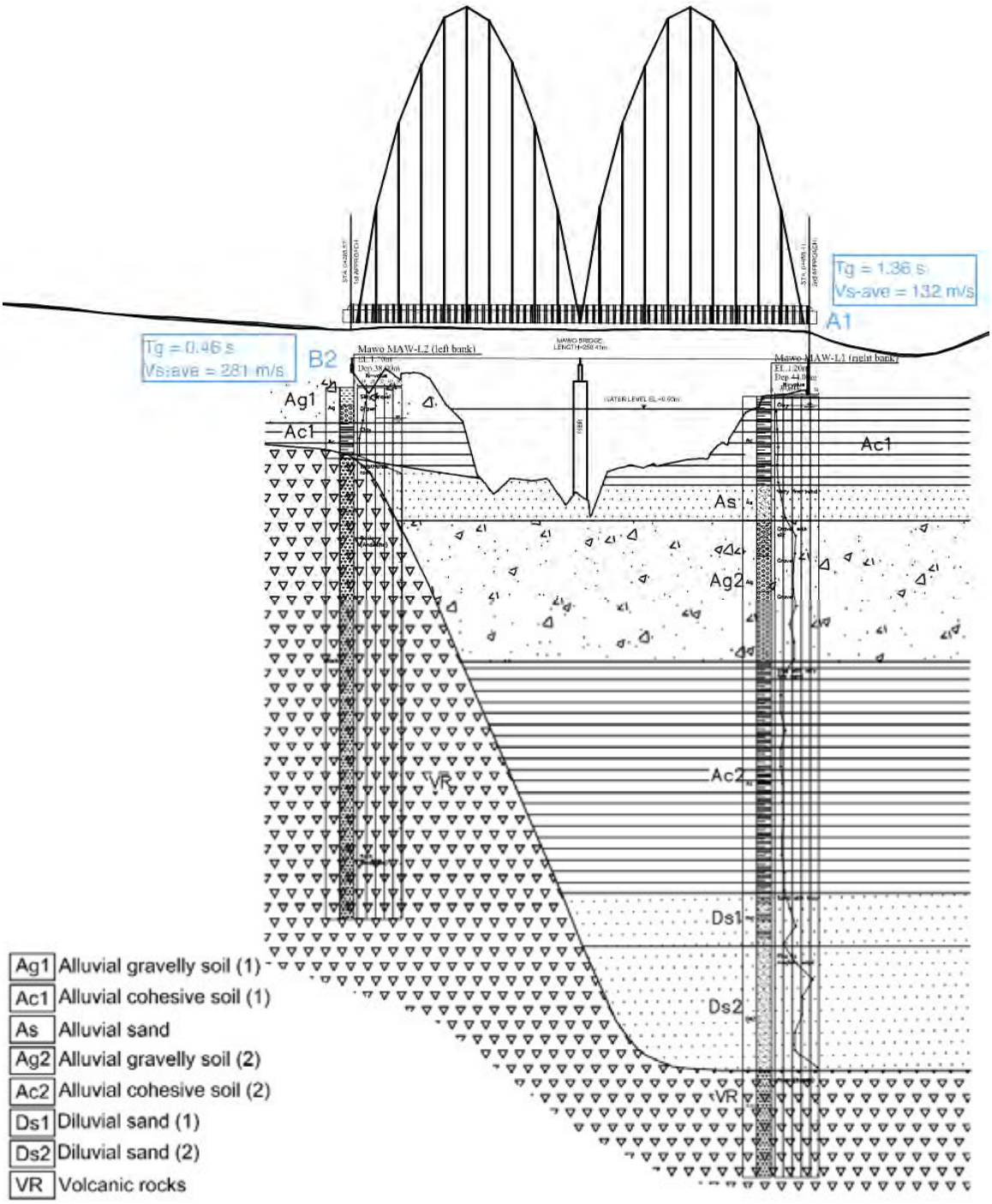


Figure 8.2-22 Site-specific design spectra for Palanit Bridge at A1



(c) soil profile

Figure 8.2-23 Mawo Bridge site data

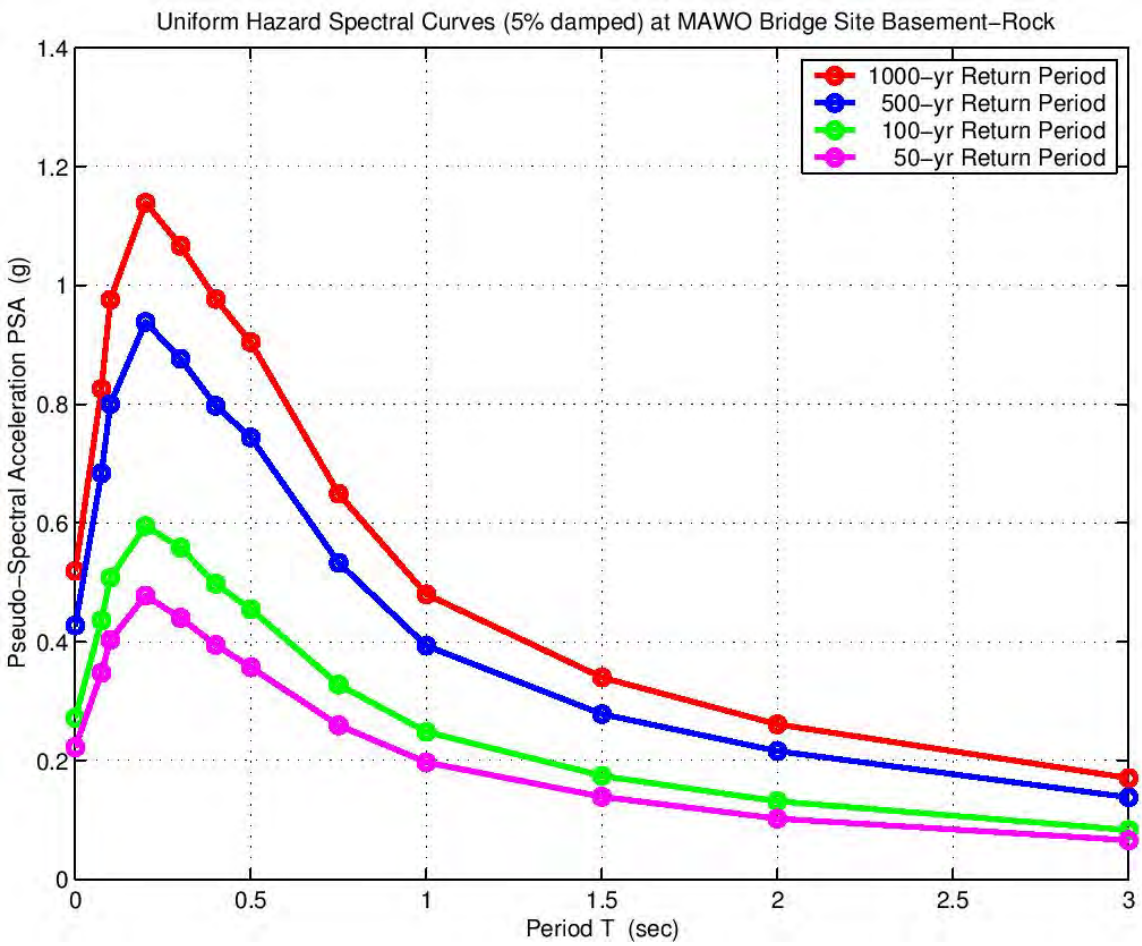


Figure 8.2-24 Uniform hazard spectral curve for Mawo Bridge site at basement-rock

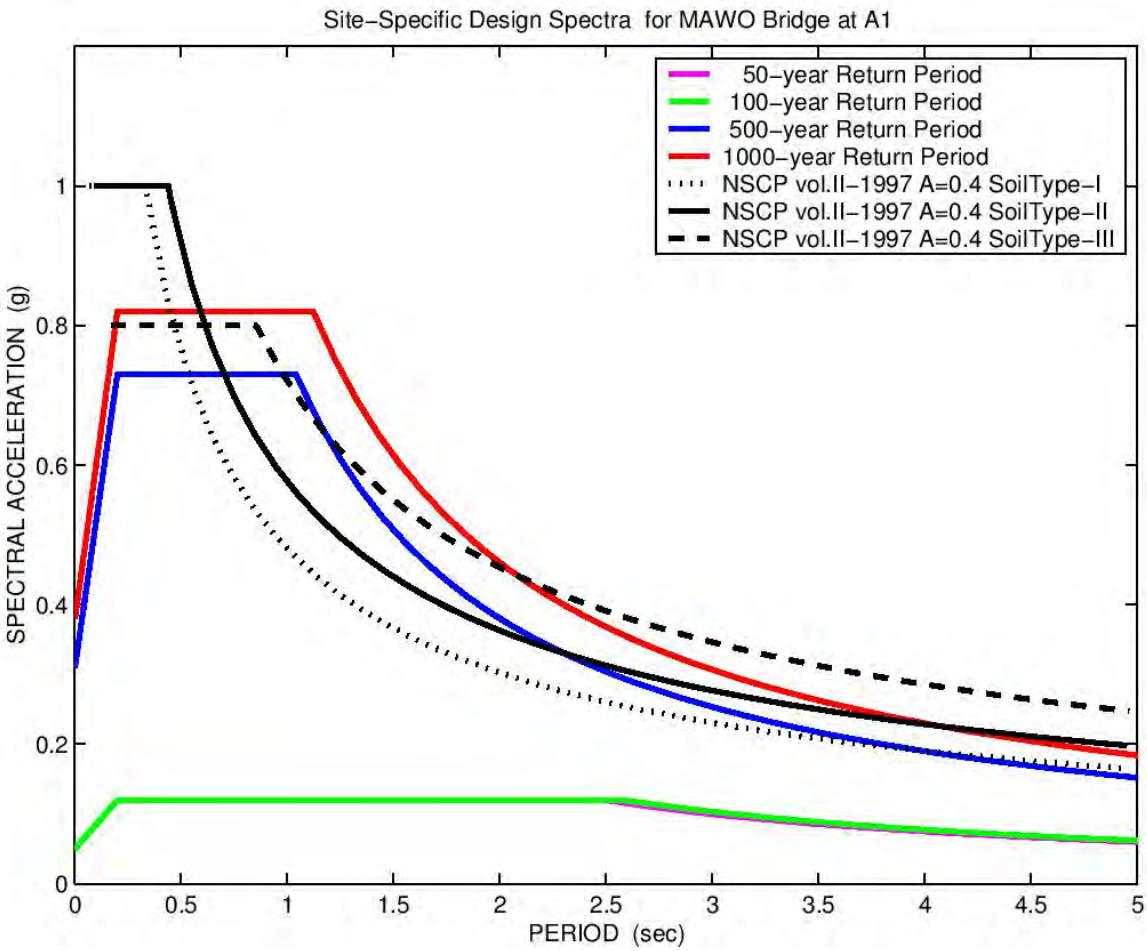


Figure 8.2-25 Site-specific design spectra for Mawo Bridge at A1

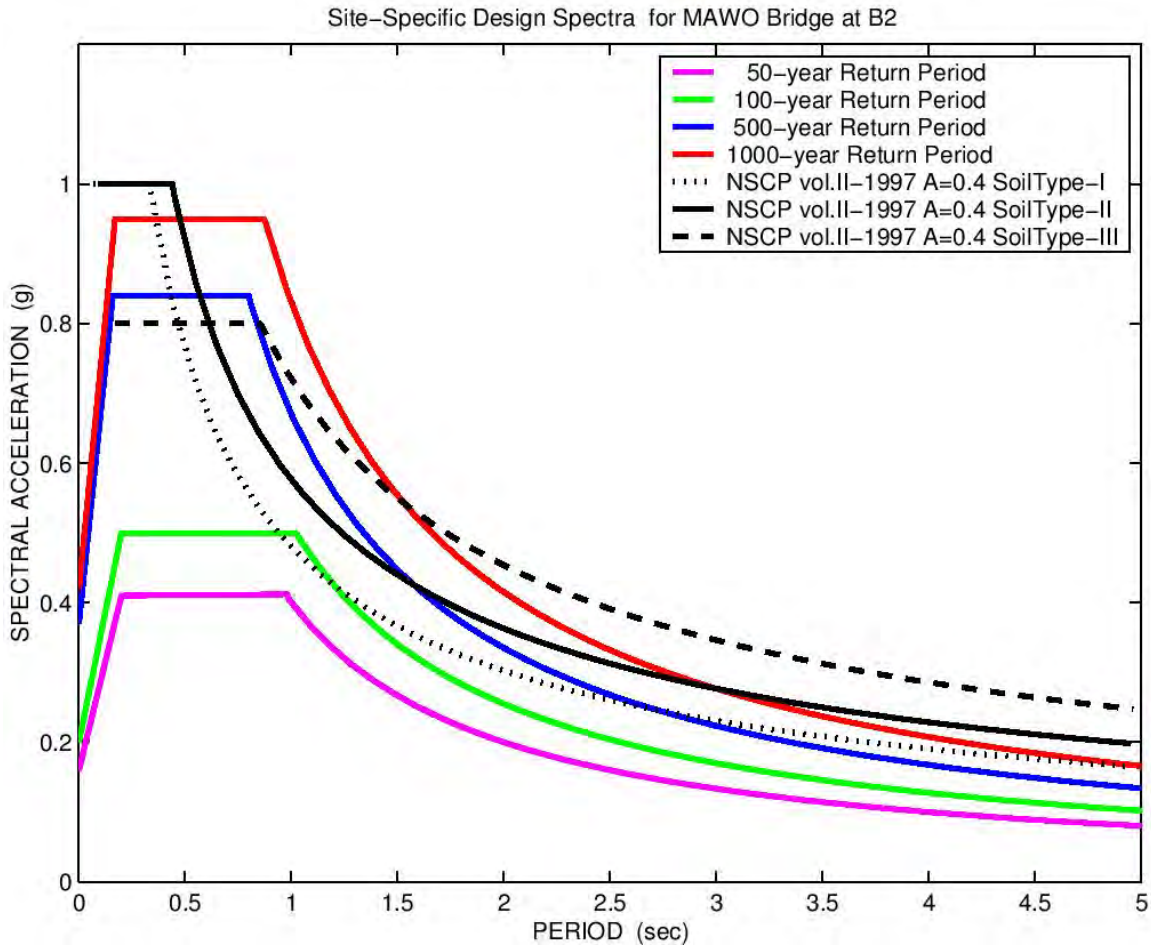
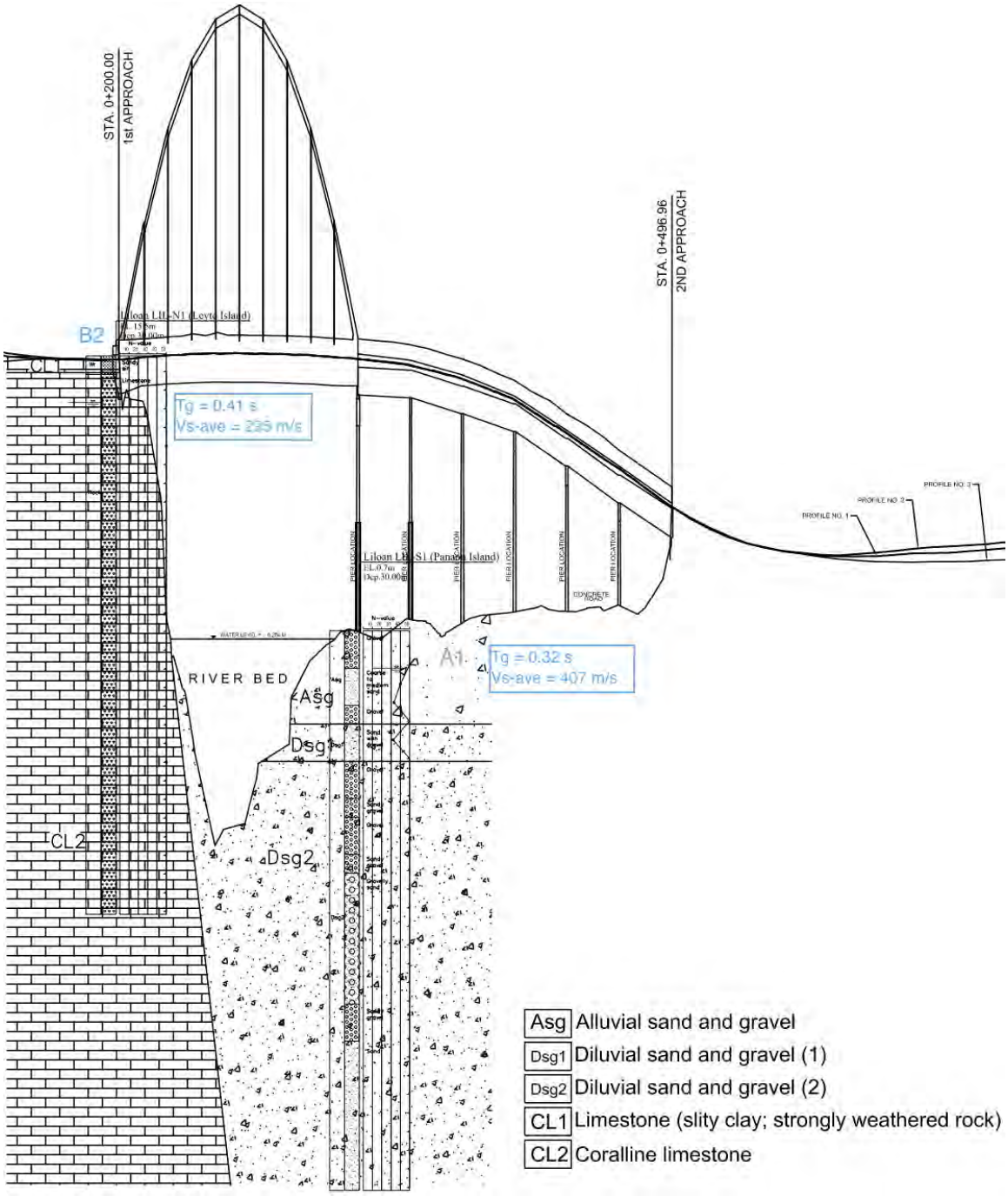


Figure 8.2-26 Site-specific design spectra for Mawo Bridge at B2



(c) soil profile

Figure 8.2-27 Liloan Bridge site data

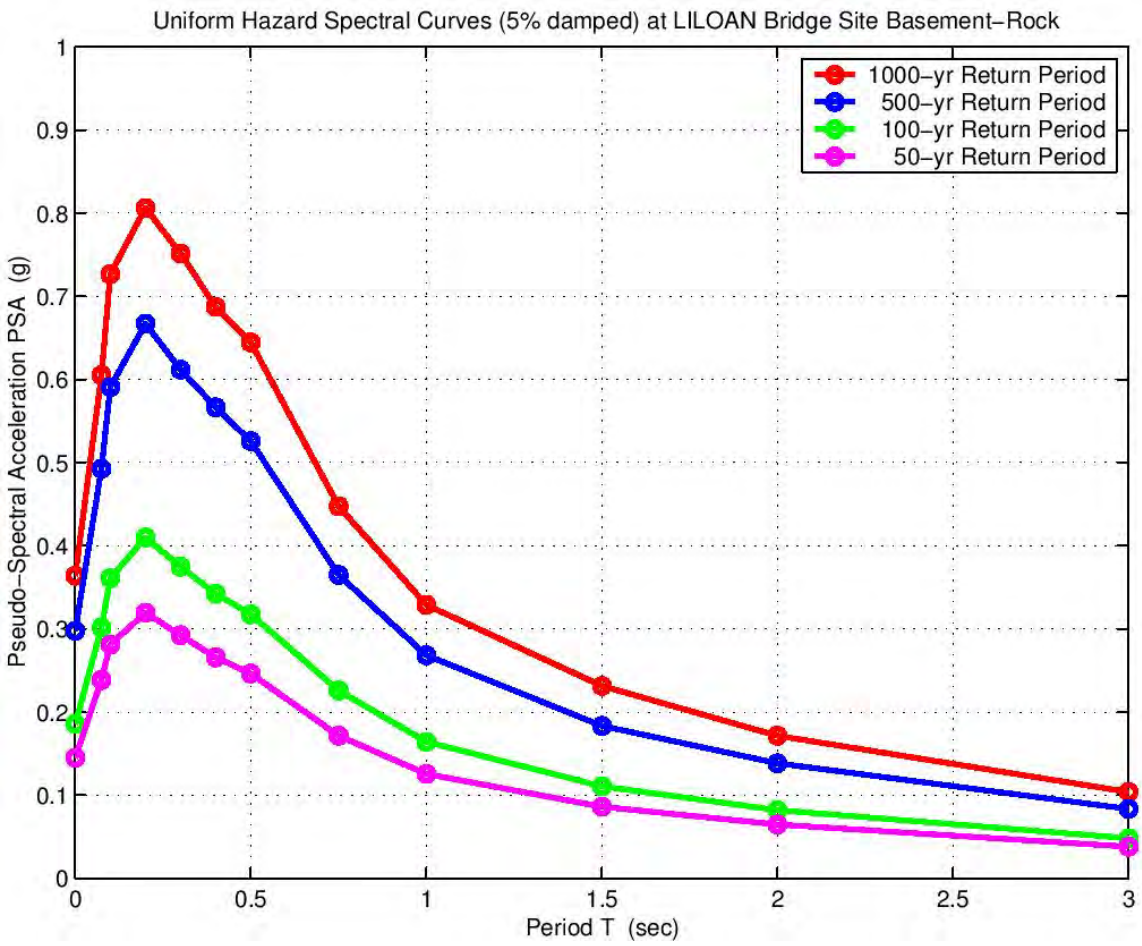


Figure 8.2-28 Uniform hazard spectral curve for Liloan Bridge site at basement-rock

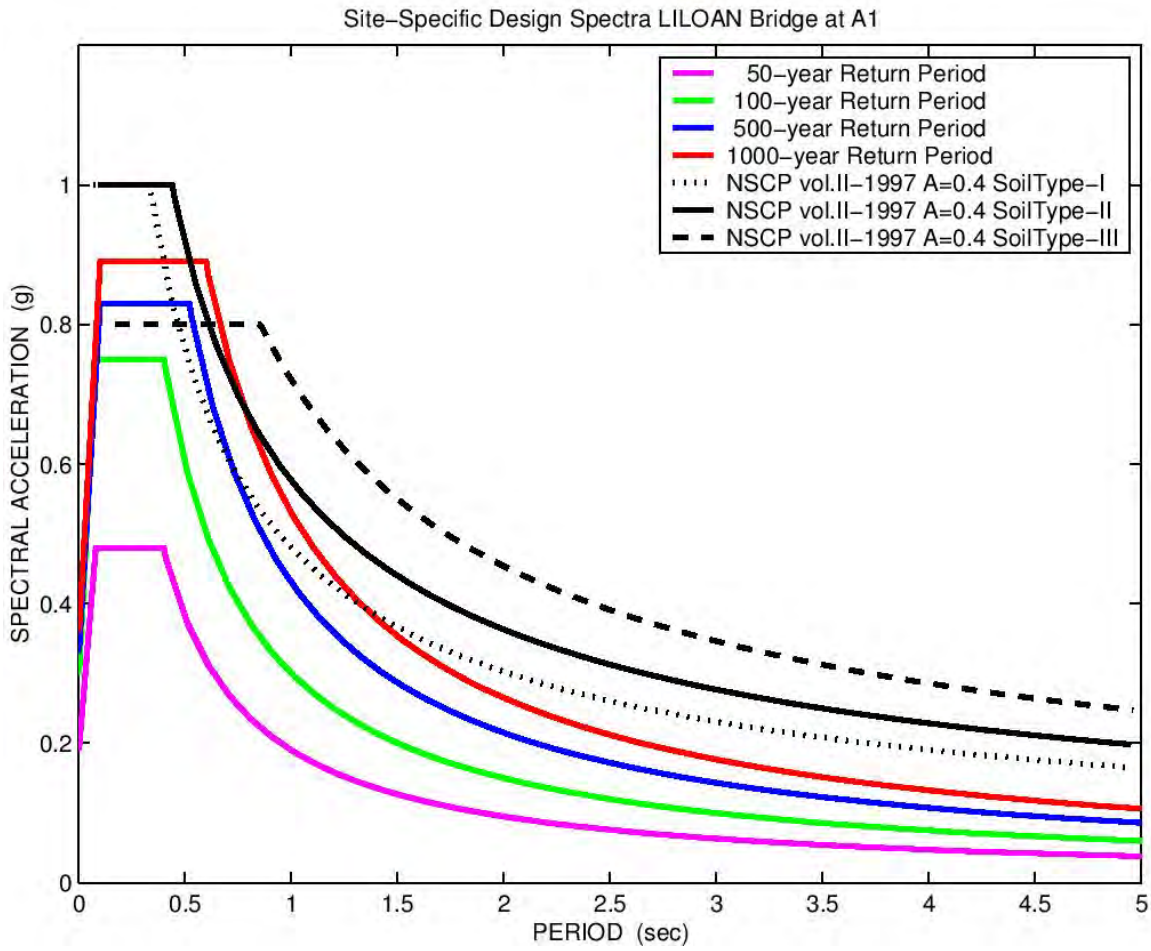
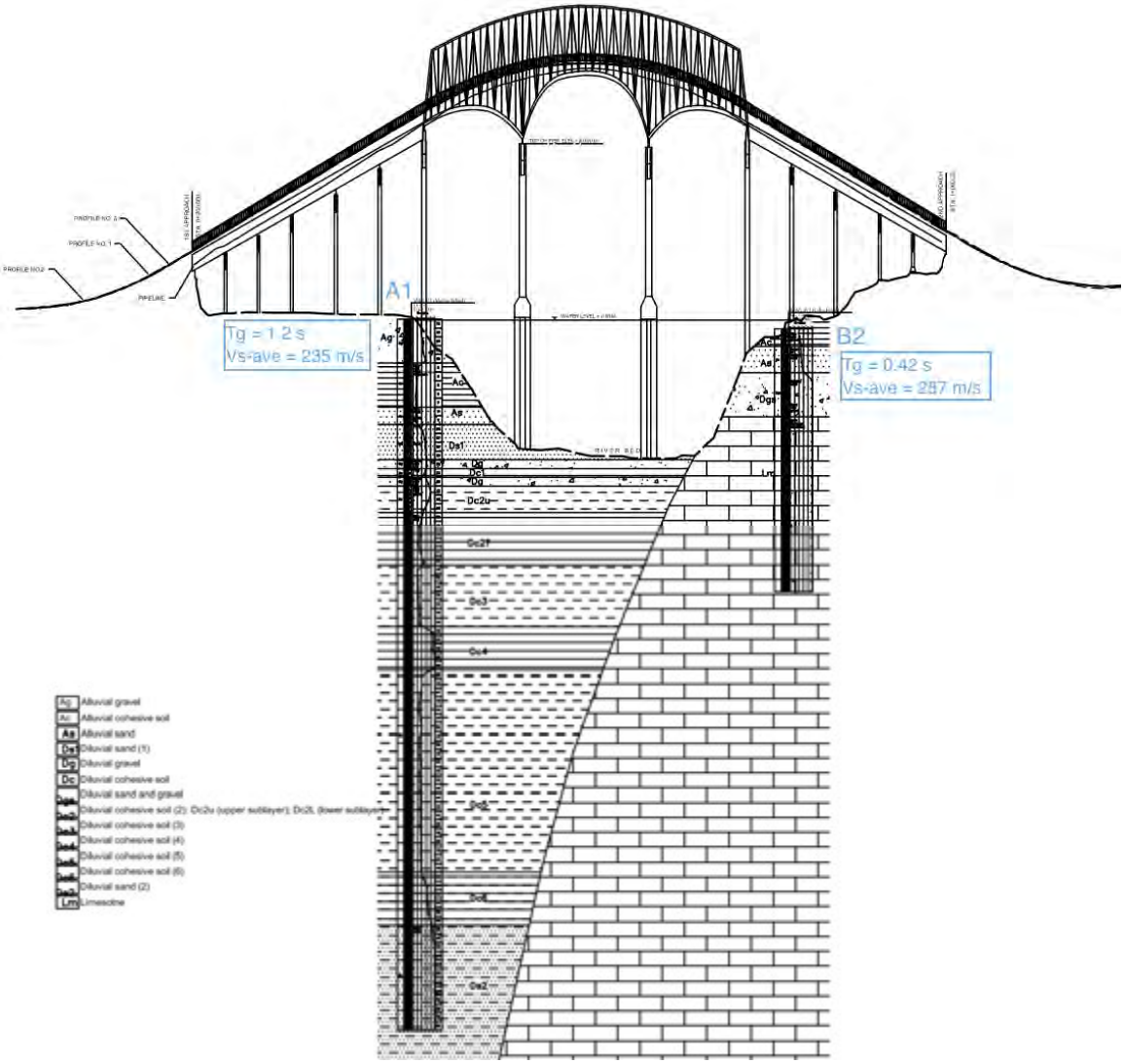


Figure 8.2-29 Site-specific design spectra for Liloan Bridge at A1



(c) soil profile

Figure 8.2-30 1st Mactan-Mandaue Bridge site data

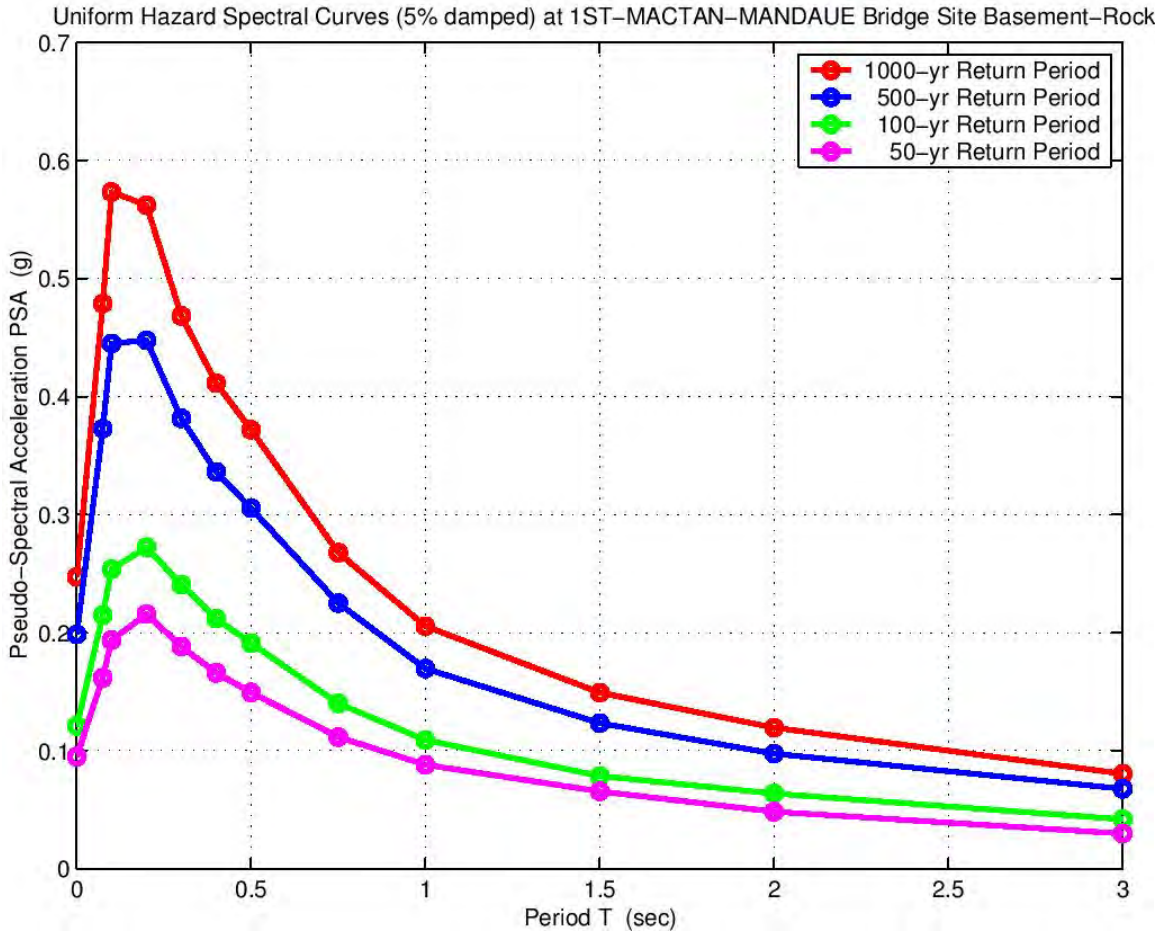


Figure 8.2-31 Uniform hazard spectral curve for 1st Mactan-Mandaue Bridge site at basement-rock

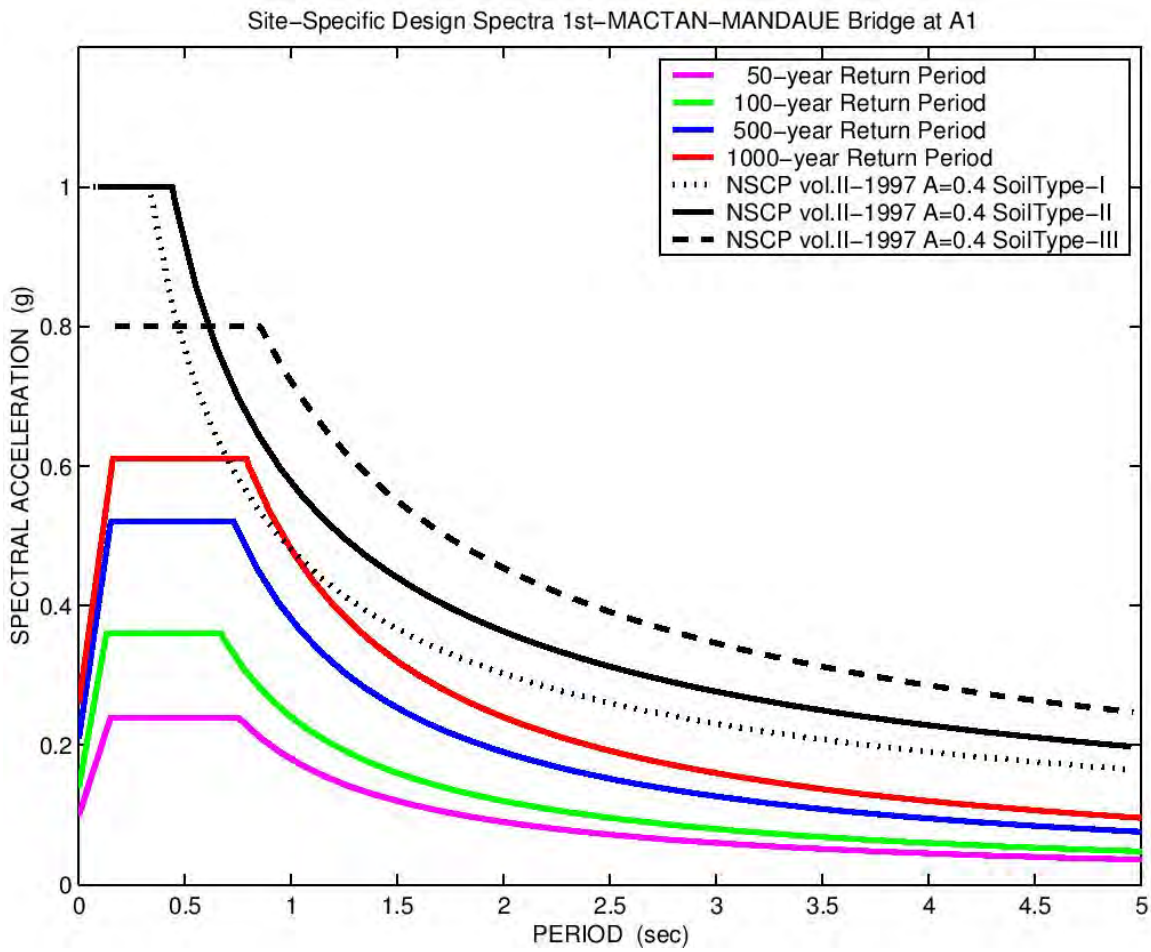


Figure 8.2-32 Site-specific design spectra for 1st Mactan-Mandaue Bridge at A1

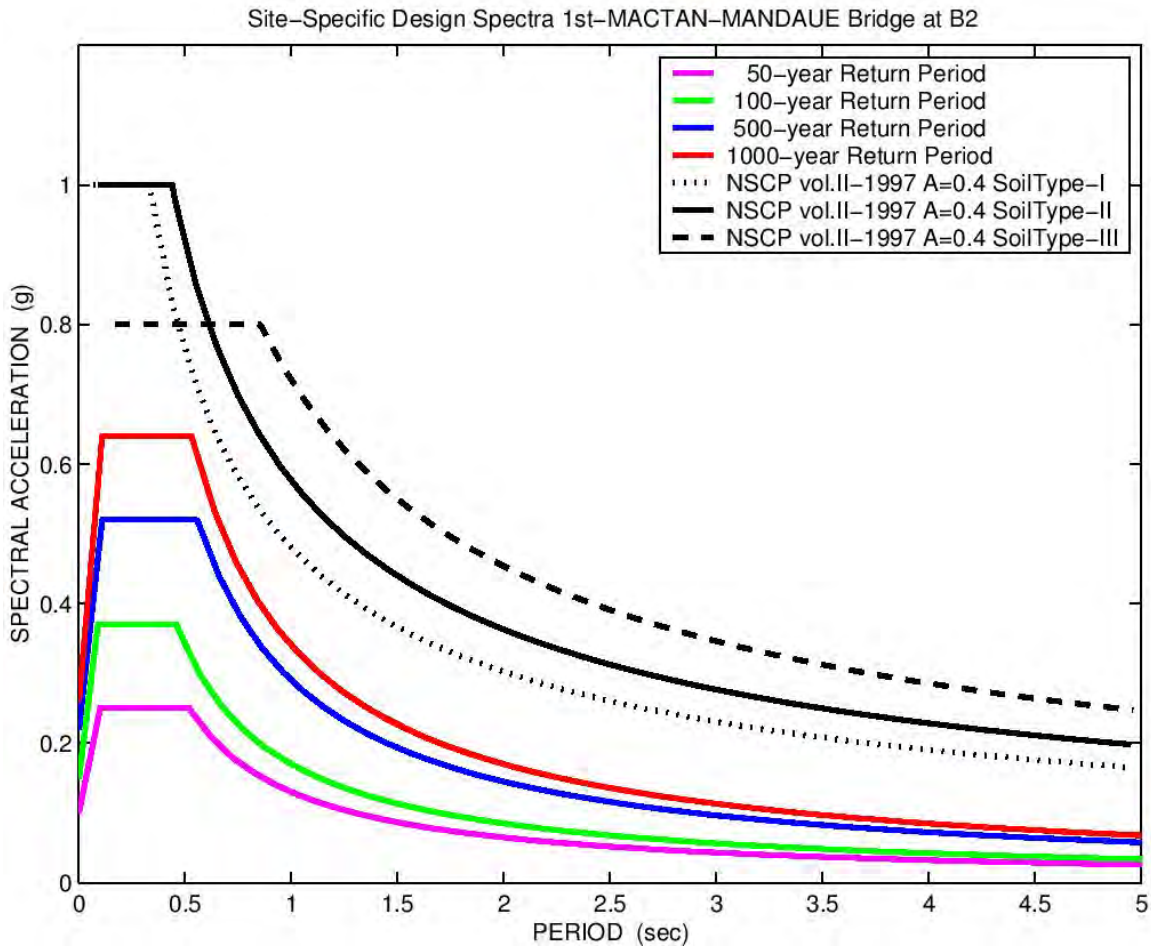
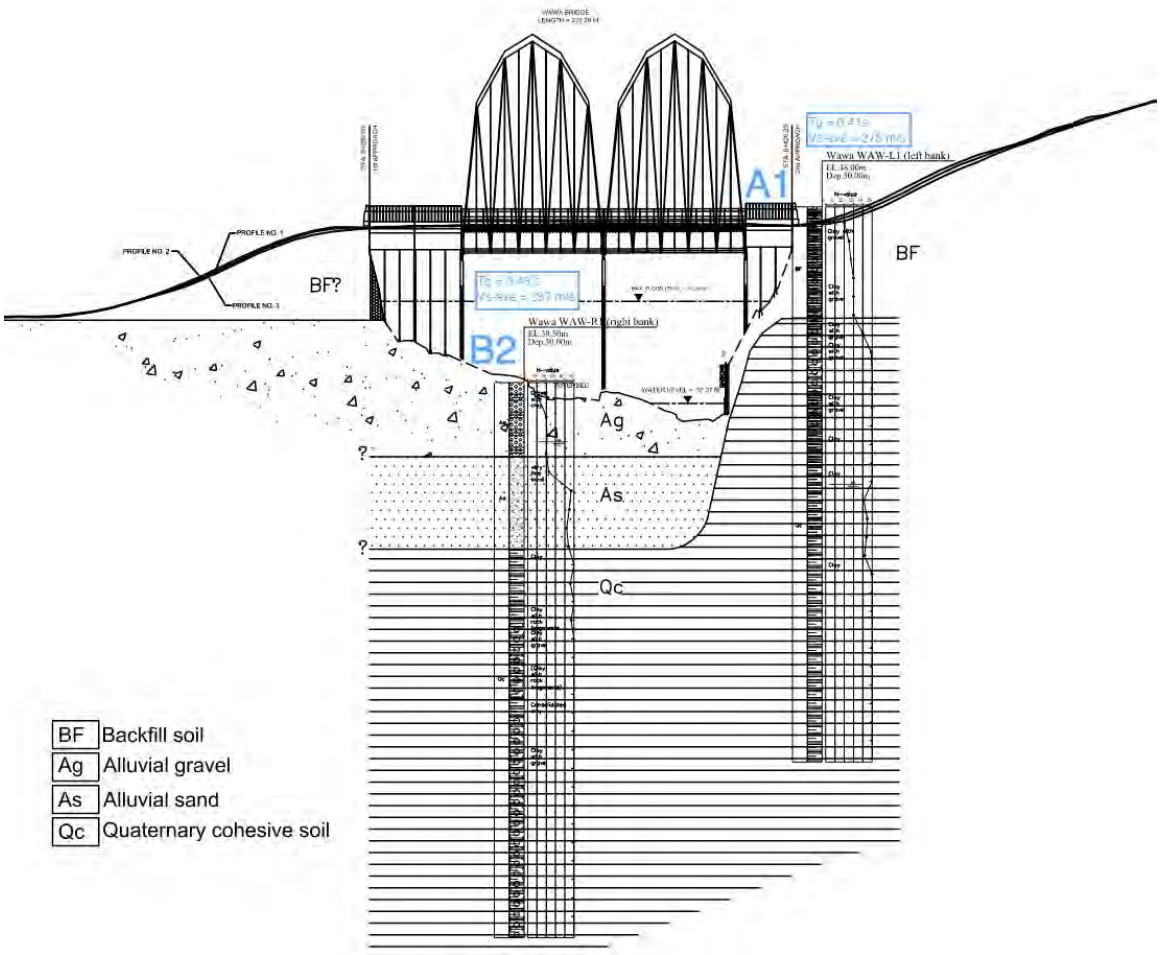


Figure 8.2-33 Site-specific design spectra for 1st Mactan-Mandaue Bridge at B2



(c) soil profile

Figure 8.2-34 Wawa Bridge site data

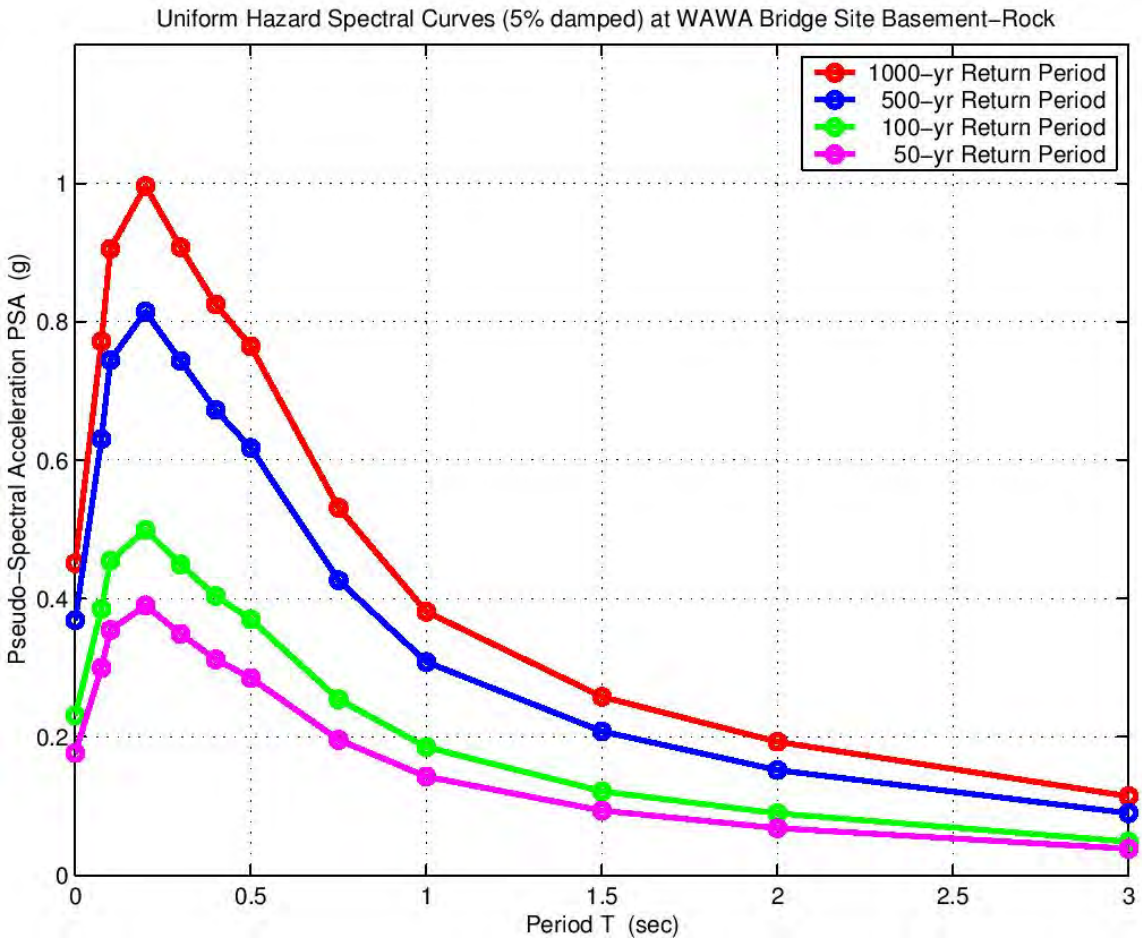


Figure 8.2-35 Uniform hazard spectral curve for Wawa Bridge site at basement-rock

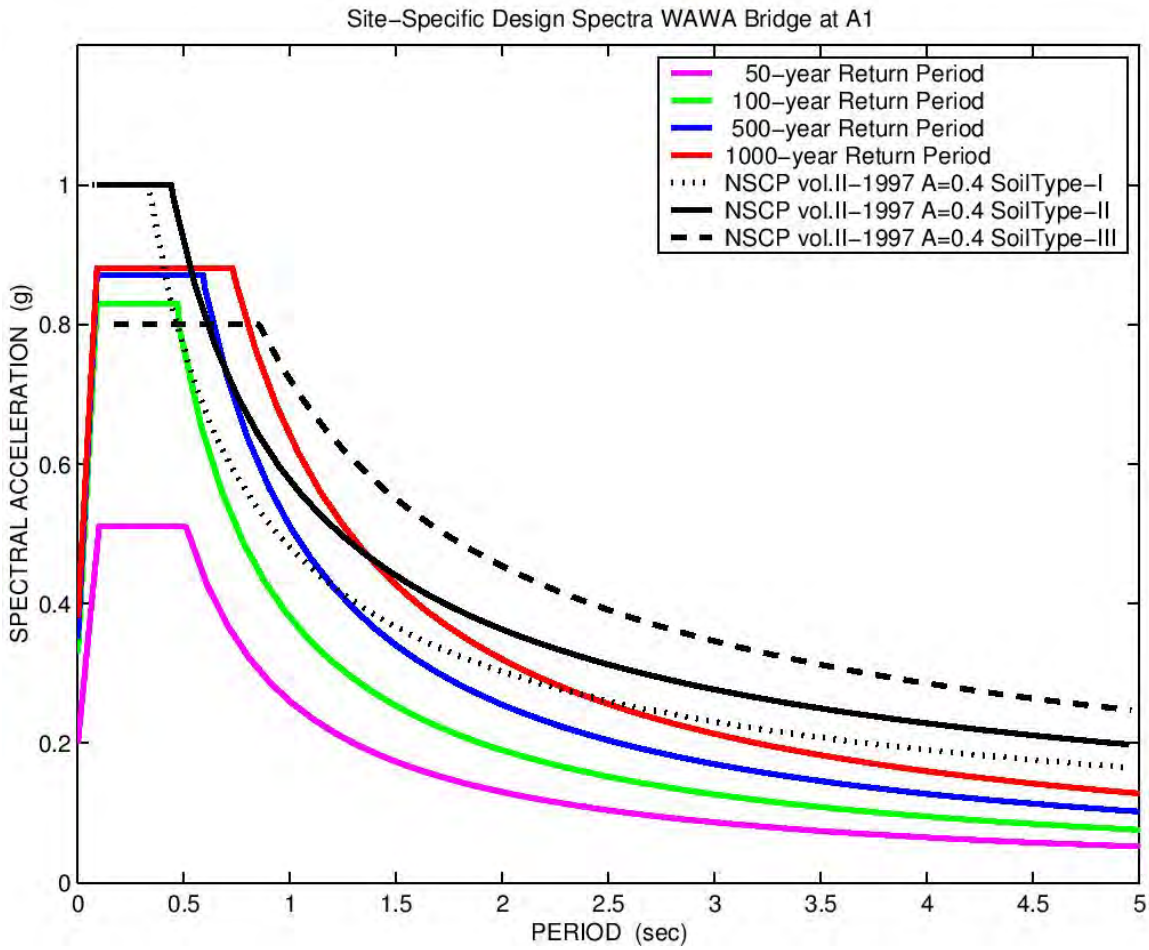


Figure 8.2-36 Site-specific design spectra for Wawa Bridge at A1

CHAPTER 9 SEISMIC HAZARD MAPS FOR DESIGN OF BRIDGES

9.1 Introduction

Earthquake loading in seismic design codes is typically specified in the form of design response spectra, most commonly the acceleration spectra. The starting point is usually a normalized or standardized spectrum defined for a reference site classification which is then multiplied by some scaling or modification factors to account for site effects (due to soil types) and potential earthquake intensity at the site.

The best way to obtain the reference spectra and their corresponding scaling factors is by analyzing extensive amount of strong-motion earthquake records obtained locally at the networks of recording stations throughout the country where the seismic code will be applied on the premise that these have recorded the largest ground motions possible for the required design level.

Currently, only Japan is able to obtain empirically its design response spectra from its extensive database of strong-motion records. Since the 1996 JRA (Japan Road Association) Seismic Design Specifications, two-level design earthquake ground motions corresponding to two-level seismic performance design philosophy are specified in which Level 2 ground motions are specified into two types: (1) Type I is the ground motion with lower (than that of Type II) probability of recurrence which could be induced in the plate boundary-type (subduction type) earthquakes with magnitude of above 8; and (2) Type II is the ground motion with higher probability of recurrence which could be developed in inland (crustal type) earthquakes with magnitudes of about 7 to 7.2 at very short distances. At the time of the introduction of new design earthquakes in the 1996 JRA Seismic Design Specifications, Type II design spectra were obtained using analyses of the numerous recordings in particular those obtained during the 1995 Hyogo-ken Nanbu Earthquake; while Type I which envisioned future earthquakes similar to the 1923 Kanto Earthquake had used numerically simulated ground motion time histories to obtained the corresponding Level-2 Type-I design spectra. These design response spectra remained the same in the revised 2002 JRA Seismic Design Specifications; and subsequently, with the availability of strong-motion records obtained during the 2011 Great East Japan (Tohoku) Earthquake, Level-2 Type-I design spectra have been updated in the most recently revised 2012 JRA Seismic Design Specifications.

Due to the non-existence or lack of comprehensive strong-motion records, the design response spectra used in the current seismic design codes of most countries (including AASHTO for highway bridges in the US) have employed probabilistic seismic hazard analysis (or combined with deterministic seismic hazard analysis) to generate design spectral parameters for obtaining the reference design response spectra to which site coefficients are applied to obtain design response spectra for a site. Given the limited knowledge and inherent uncertainties in earthquake process, probabilistic seismic hazard analysis (PSHA) enables the evaluation of the hazard of seismic ground motion at a site by considering all possible earthquakes in the area, estimating the associated shaking at the site, and calculating the probabilities of these occurrences; i.e., the evaluation of annual frequencies of exceedance of ground motion levels (typically designated by peak ground acceleration and by spectral accelerations at key periods of interest) at a site. The result of a PSHA is a seismic hazard curve (annual frequency of exceedance versus ground motion amplitude); or in recent useful form to obtain codified response spectra for a design earthquake return period, the PSHA result is a uniform hazard spectrum (spectral amplitude versus structural period for a given annual frequency of exceedance). This has been the main basis for establishing design earthquake loading in the evolution of modern performance-based seismic design codes.

Development of design earthquake ground motions for use in seismic design codes in the Philippines has been longly and largely stagnated; which has not enabled Philippine structural designers to adapt

to rapidly evolving worldwide developments in LRFD and performance-based seismic design of structures in the past 15 years or so.

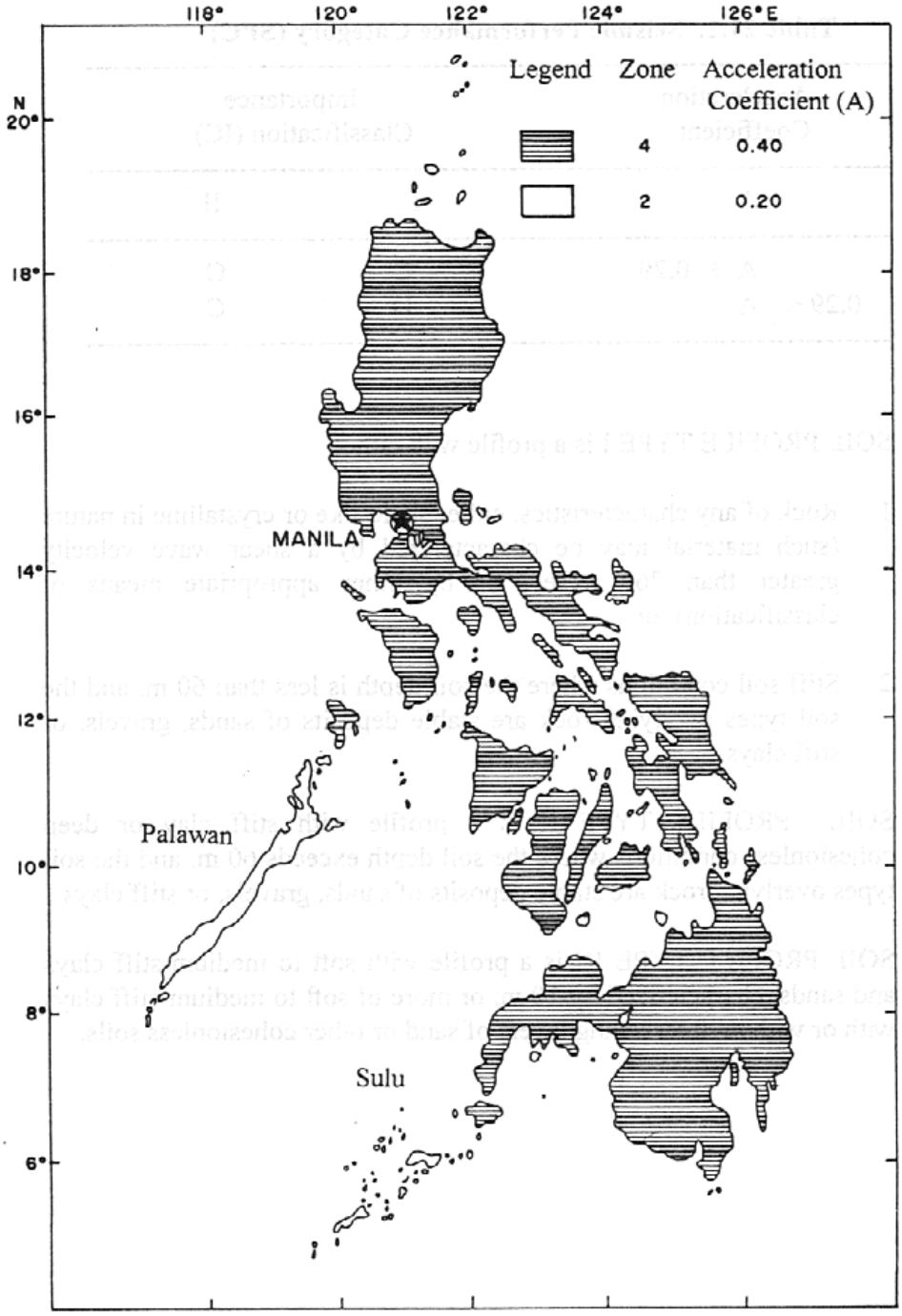


Figure 9.1-1 Seismic Zone Map of the Philippines in NSCP-bridges 1997 (also the same in NSCP-buildings 2010, 2001, 1992)

The seismic zone map in the current NSCP code for seismic design of bridges (Volume II – Bridges, 2nd edition 1997, reprinted 2007; hereafter 1997 NSCP-bridges) showed in Figure 9.1-1 gives an acceleration coefficient of 0.4 for the Philippines except for Palawan which is given an acceleration coefficient of 0.2. Similar seismic zone map is also used in the latest NSCP code for buildings (2010 NSCP-buildings) and previous two editions (2001 NSCP-buildings and 1992 NSCP-buildings). In the present usage for building code, the coefficients are made to represent the seismic hazard level expressed in terms of an acceleration coefficient (average of spectral acceleration values over the short period range) corresponding to 10% probability of exceedance in 50 years (mean return period

of 500 years). However, no documentation exists to show if a probabilistic seismic hazard analysis was ever conducted to obtain this prevailing seismic zone map.

Due to the absence of appropriate studies of design earthquake ground motion for use in updating seismic design codes, seismic design codes for buildings and bridges in the Philippines have lagged very much far behind. The seismic provisions in the current NSCP-building 2010 continues to be based on the 1997 Uniform Building Code (UBC) since the seismic mapping in the Philippines has not advanced to the requirement of spectral acceleration mapping for the so-called maximum considered earthquake (MCE) which is generally based on a 2500-year return period (2% probability of exceedance in 50 years) which is the design earthquake hazard level used since the 2000 IBC (International Building Code) which had been updated in 2003, 2006, 2009, and recently 2012 editions.

The situation is also similar in Philippine seismic design code for bridges which have traditionally adopted the AASHTO code used in the United States. Both in the latest 2012 AASHTO LRFD standards (force-based R-factor method) and the alternate 2011 AASHTO LRFD Seismic Design Guidelines (displacement-based design), design seismic maps require spectral acceleration values at PGA, at 0.2 sec, and at 1.0 sec corresponding to an earthquake hazard level with mean return period of 100 years (7% probability of exceedance in 75 years).

In the early part of the study which started in April 2012 in Manila, the JICA Study Team had recognized these deficiencies regarding the localized design earthquake ground motions for use in seismic design and the circumstances (such as, adoption of AASHTO code in the past, difficulties in adopting the latest codes, non-existence of strong-motion records in the Philippines, limited existing data in seismology are not directly usable in seismic hazard analyses).

In view of the above, the Study Team had additionally proposed to JICA to assist DPWH in initiating a scope of study on development and sustainable evolution of design earthquake motions for use in seismic design of Philippine bridges. The main objectives of the present development of design ground motions localized for Philippine bridge seismic design use are two-fold as follows:

(1) to provide design earthquake motions (in terms of design spectra and suites of time histories) for use in outline design of 2 bridges in Package-B and 5 bridges in Package-C which will commence after final evaluation based on the 2nd screening procedure;

(2) to provide contour maps of spectral parameters (PGA, SA at 0.2 sec, SA at 1. sec) corresponding to 50-year return period (78% probability of exceedance in 75 years), 100-year return period (53% probability of exceedance in 75 years), 500-year return period (14% probability of exceedance in 75 years) and 1000-year return period (7% probability of exceedance in 75 years) to furnish DPWH with options on Level 1 and Level 2 design earthquake motions needed to advance to state-of-the-art seismic design code for bridges with considerations of local seismic hazards and economic loss acceptability.

To achieve these objectives, the scope of work includes:

(1) Constitute a process-framework to jumpstart the probabilistic seismic hazard analyses in this study and would be sustainable for future upgrading and updating;

(2) Develop site-specific design spectra for 7 bridge sites (2 bridges in Package B and 5 bridges in Package C);

(3) Develop contour maps of PGA and spectral accelerations at 0.2 sec and 1.0 sec for AASHTO site class B (equivalent to Vs30 = 760 m/s) corresponding to return periods of 50 years, 100 years, 500 years and 1000 years.

9.2 Methodology and Return Periods

Probabilistic seismic hazard analysis (PSHA) provides a framework in which uncertainties in the size, location, and rate of recurrence of earthquakes and in the variation of ground motion characteristics with earthquake size and location can be identified, quantified, and combined in a rational manner (Thenhaus and Campbell, 2003).

The probability that an observed ground motion parameter X (spectral acceleration, in this study) will be greater than or equal to the value x in the next t years (the exposure period) given the annual exceedance rate $\lambda\,[X \geq x]$ is computed as:

$$P\,[X \geq x] = 1 - \exp\,(-t\,\lambda\,[X \geq x])$$

$$\lambda\,[X \geq x] \approx \sum_{\text{sources } i} v_i \int_{m_0}^{m_{max}} \int_{R|M} P\,[X \geq x \mid M, R]\, f_M(m)\, f_{R|M}(r|m)\, dr\, dm$$

where

$\lambda\,[X \geq x]$	the annual frequency that ground motion at a site exceeds the chosen level $X = x$;
v_i	the annual rate of occurrence of earthquakes on seismic source *i* having magnitudes between m_o and m_{max};
m_o	the minimum magnitude of engineering significance (taken to be 5.0 in this study);
$P\,[X \geq x \mid M, R]$	the conditional probability that the chosen ground motion level is exceeded for a given magnitude *M* and distance *R*;
$f_M(m)$	probability density function of Earthquake magnitude;
$f_{R|M}(r|m)$	probability function of distance from the earthquake source to the site of interest.

In hazard risk analysis, a return period is an estimate of the time interval between hazard events of similar severity (exceeding a certain intensity or size); i.e., spectral acceleration parameters for earthquake ground motion and structural response used in our study. In practice, the terms “return period” and “recurrence interval” have been interchangeably used (Abrahamson 2000). Strictly speaking, the recurrence interval is the time interval between earthquakes of given magnitude or larger for on a seismic source; whereas the return period refers to the reciprocal of the annual rate at which a ground motion level is exceeded at a site.

Return period should not be interpreted as to what may occur over the implied period of time; but should be properly interpreted as the implied period of time that the ground motion at the site has the chance (reciprocal of the annual exceedance rate) of being exceeded.
Design earthquake hazard levels are most usually expressed in terms of peak ground acceleration or

spectral response acceleration with certain return periods or equivalently in terms of the certain probability of exceedance in a given design exposure period.

The "probability of exceedance" represents the probability (expressed as percentage) that the ground motion level will be exceeded within a specified exposure time (expressed in years). Assuming that the temporal occurrence of the earthquake follows a Poisson process, the probability of exceedance of y* (say, peak ground acceleration) in a time period T_e is:

$$P[Y_T > y^*] = 1 - e^{-\lambda_{y*} T_e}$$

where λ_{y*} is the annual rate of exceedance of y^*; and return period which is the number of years between exceedances can be computed as $T_R = 1/\lambda_{y*}$.

The choice of exposure time may be somewhat arbitrary and often associated with the design life span of the structure although the actual service life span of the structure may be longer. Normally, it is taken as 50 years for buildings and 75 years for bridges. In structural design for buildings (e.g., FEMA 356, ASCE 41-06), an earthquake hazard level with "frequent" earthquakes is one with earthquake motions having a return period of 75 years (or 50% probability of exceedance in 50 years); an earthquake hazard level of "rare" earthquakes is one with earthquake motions having a return period of 225 years (or 20% probability of exceedance in 50 years); an earthquake hazard level of "very rare" earthquakes is one with earthquake motions having a return period of 475 years (or 10% probability of exceedance in 50 years); and an earthquake hazard level of "extremely rare" earthquakes is one with earthquake motions having a return period of 2500 years (or 2% probability of exceedance in 50 years).

On the other hand, design earthquake (1000-year return period) as defined in the 2010 AASHTO LRFD Bridge Design Specifications and later (2012), as well as in the AASHTO Guide Specifications for LRFD Seismic Bridge Design 1st ed. (2009) and 2nd ed. (2011), is taken for earthquake ground motion having a probability of exceedance of 7% in 75 years. Prior AASHTO LRFD Specifications (2007 or earlier) referred to 475-year return period earthquake ground motion as one having a 10% probability of exceedance in 50 years.

Performance-based seismic design philosophy in modern seismic design codes takes seismic vulnerability of a structure as to be determined by the risk associated with the design earthquake hazard level and the specified performance criteria. Determining what is an acceptable risk for a class of infrastructure (say, highway bridges) in a particular country is a very important task for the authority (DPWH) that must consider both social and economic aspects of the country.

In the following, design return periods of some current seismic design codes are described in order to provide DPWH with references in establishing the design return period for Level-2 earthquake motions that will be introduced in the draft bridge specifications that will be a product of this study.

The return period and performance requirements of the current AASHTO specifications (both the R-factor force-based 2012 AASHTO LRFD Bridge Specifications and the displacement-based 2011 AASHTO LRFD Seismic Guidelines) specify that bridges shall be designed for the objective of life safety performance considering a seismic hazard corresponding to 7% probability of exceedance in 75 years, which is equivalent to a return period of about 1,000 years; moreover, AASHTO allows bridge owners to authorize higher performance levels, such as 2500-year return period for operational objective of critical bridges, if the need to be established is required.

Since JRA used voluminous number of actual strong-motion records to obtain the design acceleration response spectra, the design earthquakes are not directly associated with equivalent return periods. However, we could surmise that Level-1 could probably be associated with 50_100-year return period; while Level-2 could be associated with 1,000_2,000-year return period.

Until November 2009, CALTRANS (California Department of Transportation) had used deterministic maximum credible earthquake for design of ordinary standard highway bridges in California. In its current Seismic Design Criteria (2010 SDC), CALTRANS has required both the deterministic maximum credible earthquake and the probabilistic acceleration response spectral curves with 5% probability of exceedance in 50 years (equivalent to a return period of 1,000 years), whichever is greater.

For seismic design of railway infrastructures, AREMA (American Railway Engineering and Maintenance-of-Way Association) has performance criteria: serviceability level (level-1, 50 to 100 years return period), ultimate level (level-2, 200 to 500 years return period) and survivability level (level-3, 1000 to 2400 years return period). As for port facilities, the Port of Long Angeles and the Port of Long Beach have adopted their performance levels of seismic design: operational level event (OLE) corresponding to 72 years return period, contingency level event (CLE) corresponding to 475 years return period, and design event (DE) corresponding to two-thirds of 2475 year probabilistic value or two-thirds of 150% of deterministic value, whichever is lower.

This PSHA study with comprehensive considerations of Philippine seismicity and geologic conditions will provide DPWH with design earthquake motions with a choice between return periods of 50 years and 100 years which will be recommended as Level-1 design earthquake motion; and two design earthquake motions corresponding to higher design periods of 500 years and 1,000 years in order to provide DPWH with options to select for Level-2 design earthquake motion in the draft bridge specifications with considerations of social and economic aspects.

9.3 Proposed Generalized Seismic Hazard Maps for the Design of Bridges — Coefficients of PGA, 0.2-sec Acceleration Response and 1.0-sec Acceleration Response

Code normally specifies design response spectrum for the design of the bridge. The design response spectrum for a specific bridge varies depending on geographic location (due to the effect of different earthquake generators and their respective distances to the site) and site characteristics (due to local site effects).

Code provides a series of ground motion intensity maps and amplification factors to account for differences in regional seismicity and site conditions. Also prescribed is a response spectrum shape and rules to construct the required design spectra in terms of given key spectral parameters and to modify the response spectra for local site effects.

This part of the PSHA study is performed to generate seismic acceleration spectral maps of the Philippines for the draft BSDS for the following return periods: (1) 1,000 years; (2) 500 years; (3) 100 years; and (4) 50 years. At each return period, spectral maps are developed for three key spectral acceleration parameters: (1) peak ground acceleration (PGA); (2) spectral acceleration at 0.2 sec; and spectral acceleration at 1. sec. The maps are developed for AASHTO site class B (equivalent to Vs30 = 760 m/s. The flow procedure is shown in Figure 9.3-1 .

Active faults as presently identified by PHIVOLCS are shown plotted in Figure 9.3-1 . Earthquake events from 1907 to 2012 (plotted in Figure 9.3-2) that were instrumentally recorded are compiled in the earthquake catalog. The magnitude scale is homogenized in a common moment magnitude scale. Declustering algorithm is applied to retain only independent main shocks (as shown plotted in Figure 9.3-3), removing aftershocks and foreshocks. Seismic source modeling consisting of fault models and background seismicity models are shown in Figure 9.3-2.

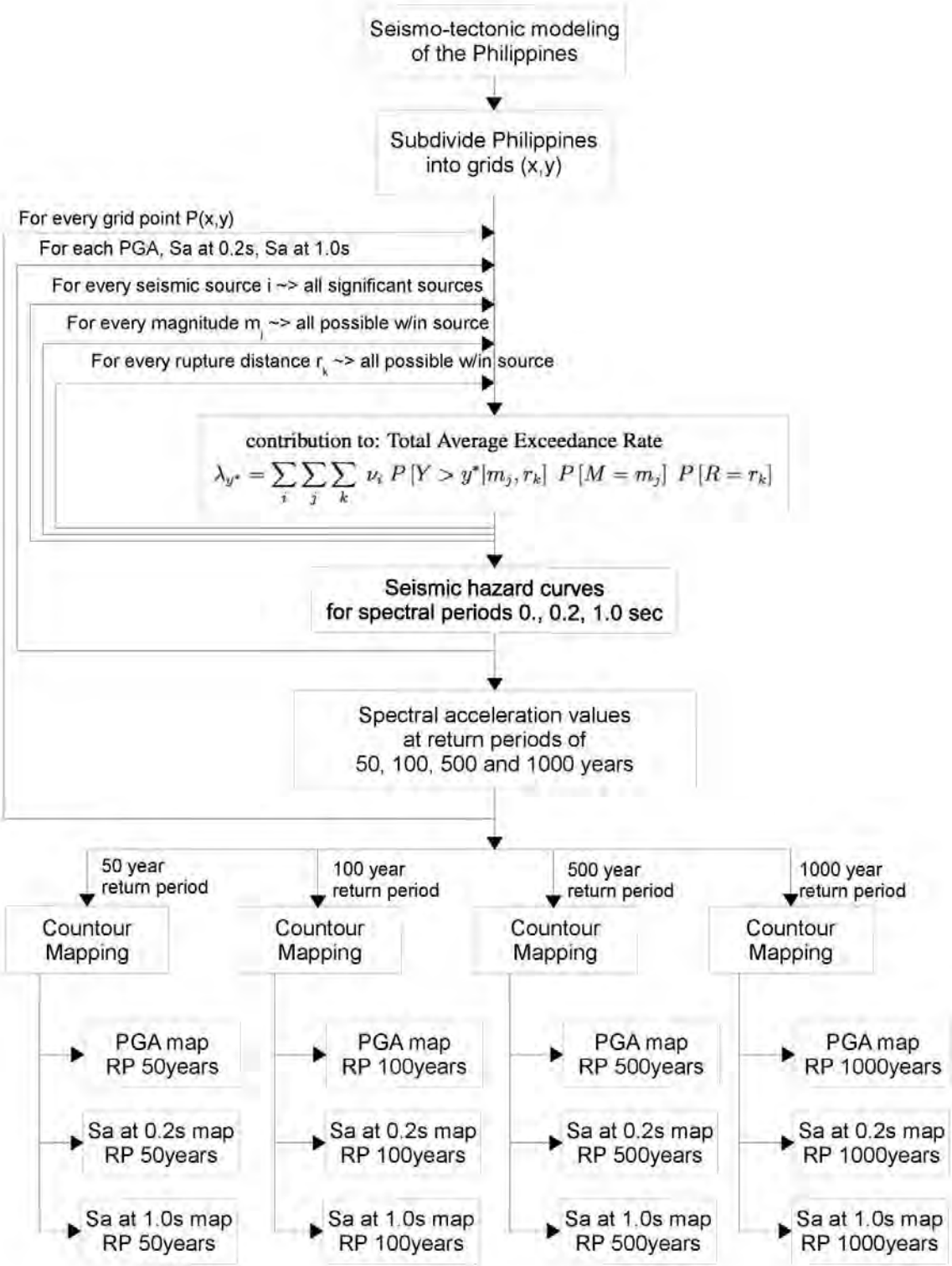


Figure 9.3-1 Procedure of PSHA Study for Spectral Mapping of PGA, Sa at 0.2 s and 1.0s at Base Rock Equivalent to AASHTO Site Class B (Vs30 = 760 m/s) Corresponding to Return Periods of 50, 100, 500, and 1000 years

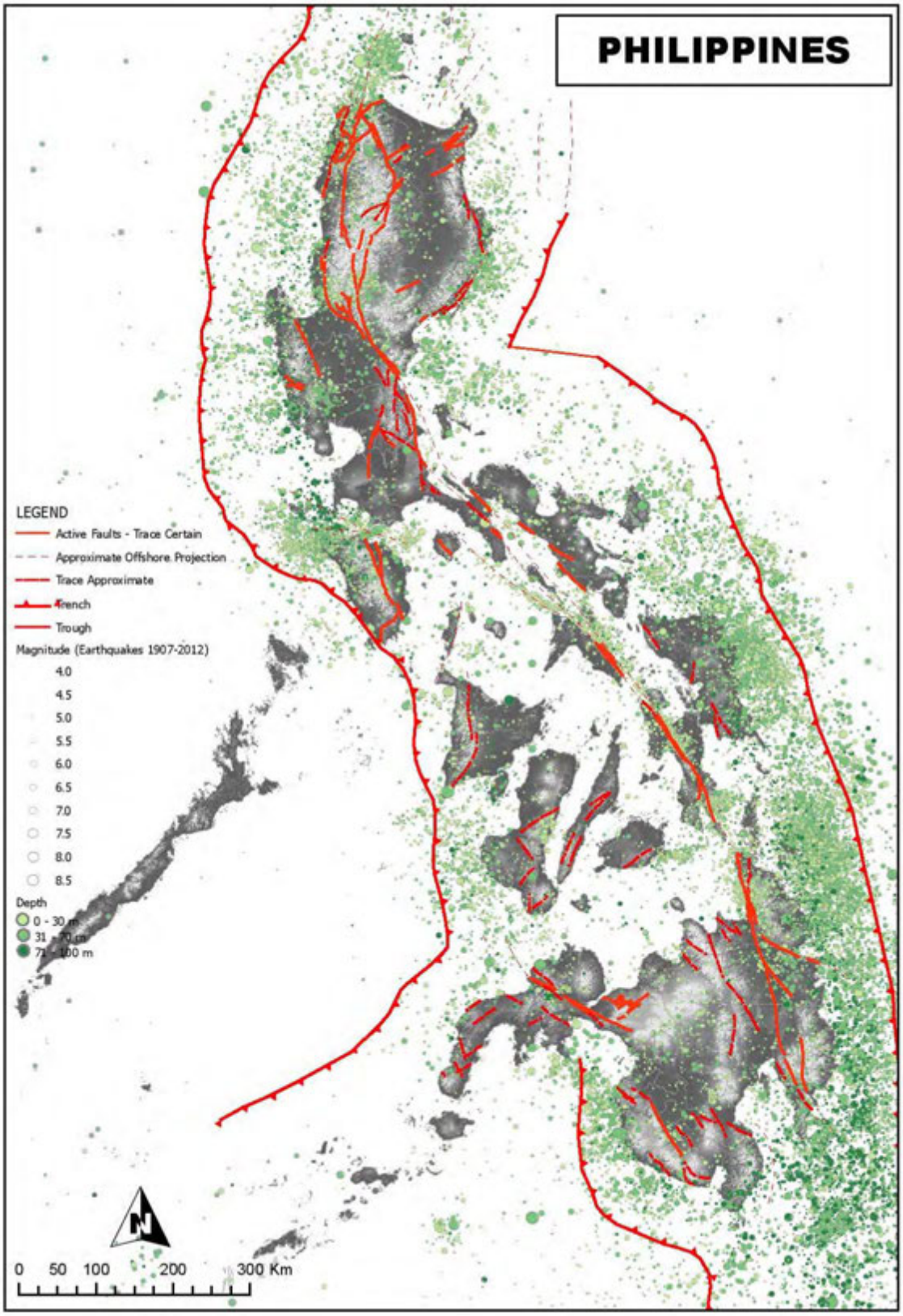


Figure 9.3-2 Seismological and Tectonic Setting of the Philippines

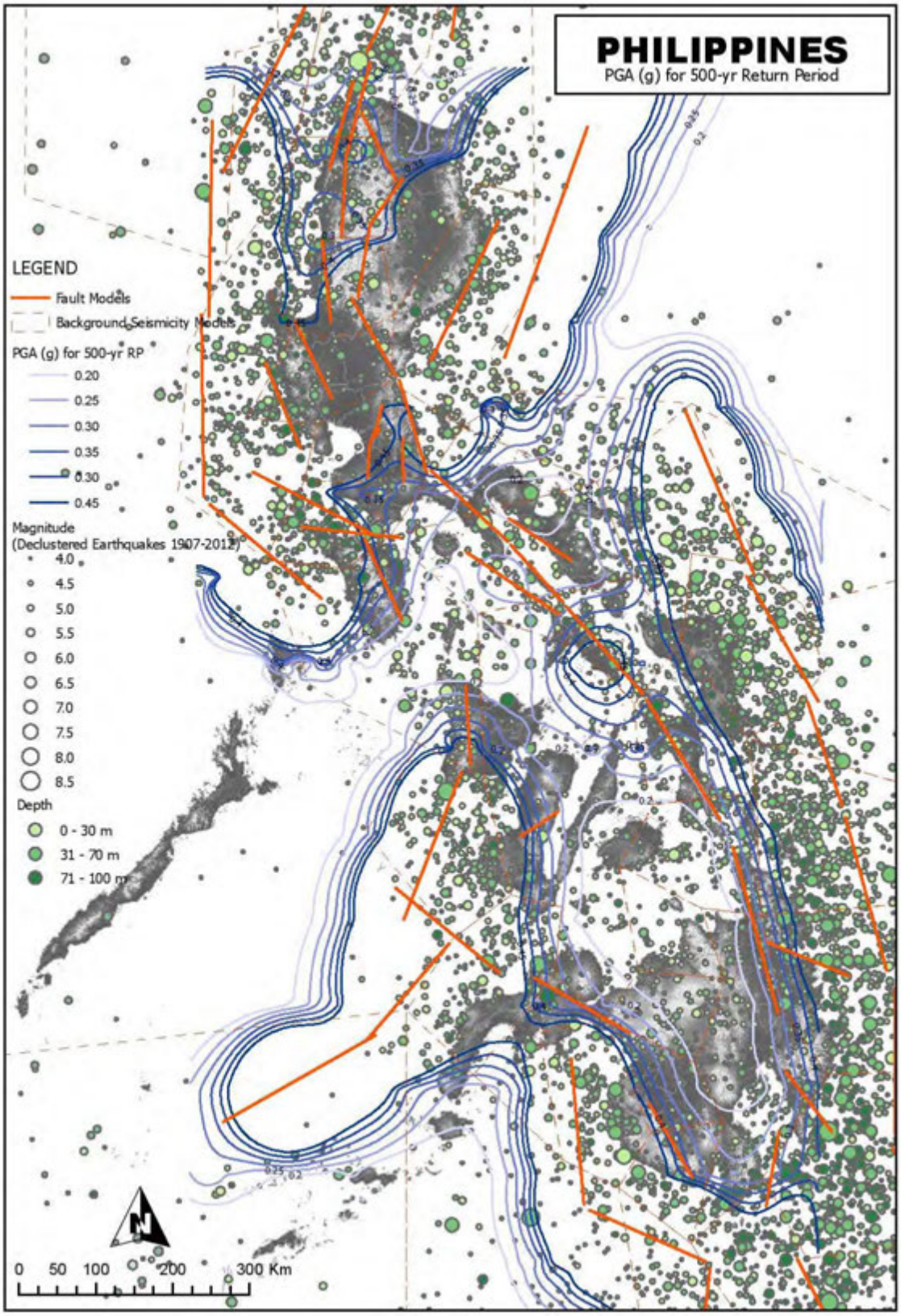


Figure 9.3-3 Seismic Source Modeling (Fault Models and Background Seismicity Models) for this PSHA Study of the Philippines

The iterative analysis is carried out for a grid interval of 10 km covering the whole Philippines for a total of 16,471 points. Interpolation and smoothing of the contours are made using the nearest-neighbor algorithm.

Figure 9.3-4 to Figure 9.3-16 present the seismic acceleration spectral maps which constitute four (corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years) sets of spectral maps at 3 key periods (0. sec, 0.2 sec, and 1. sec).

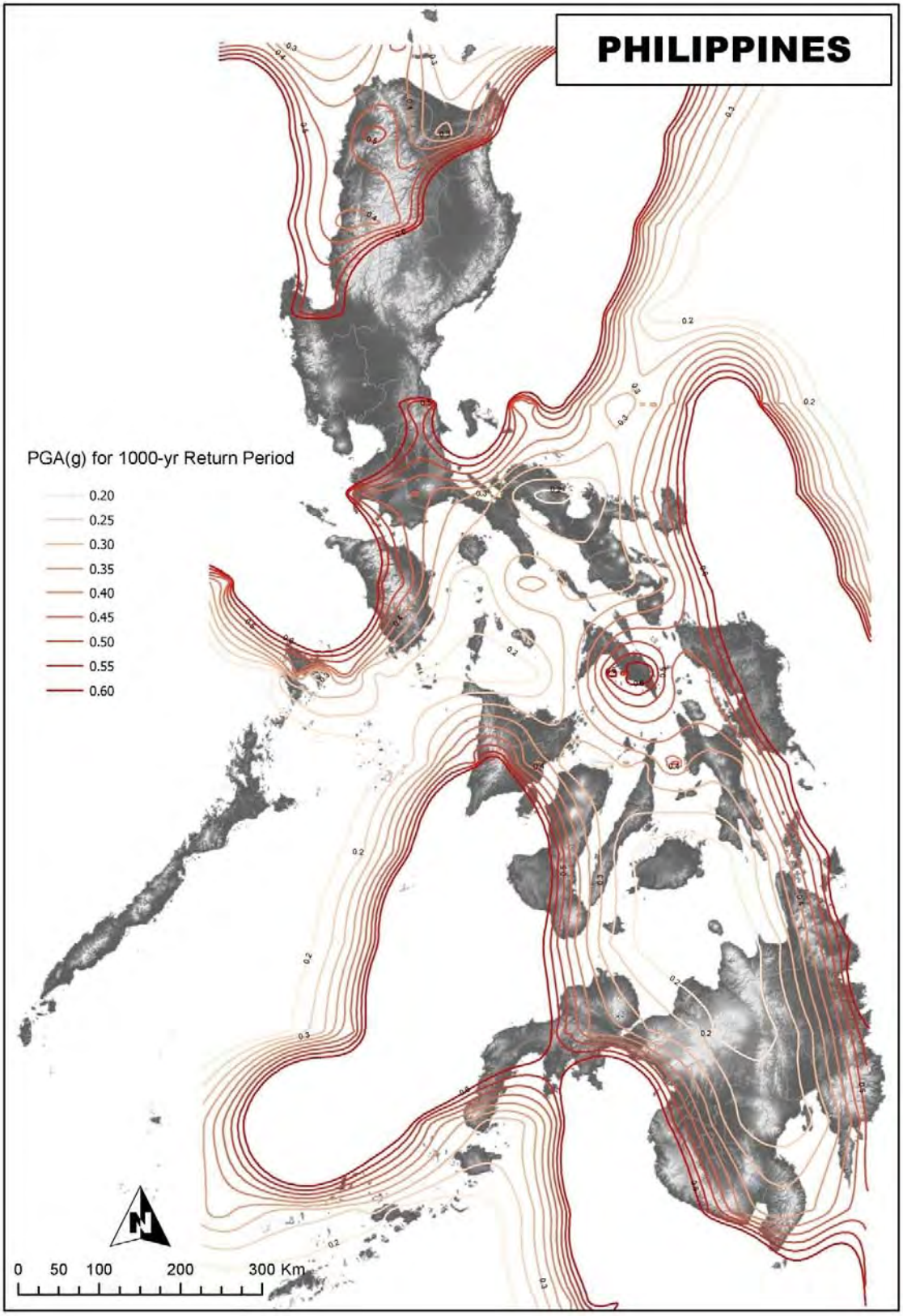


Figure 9.3-4 PGA for 1,000-year Return Period

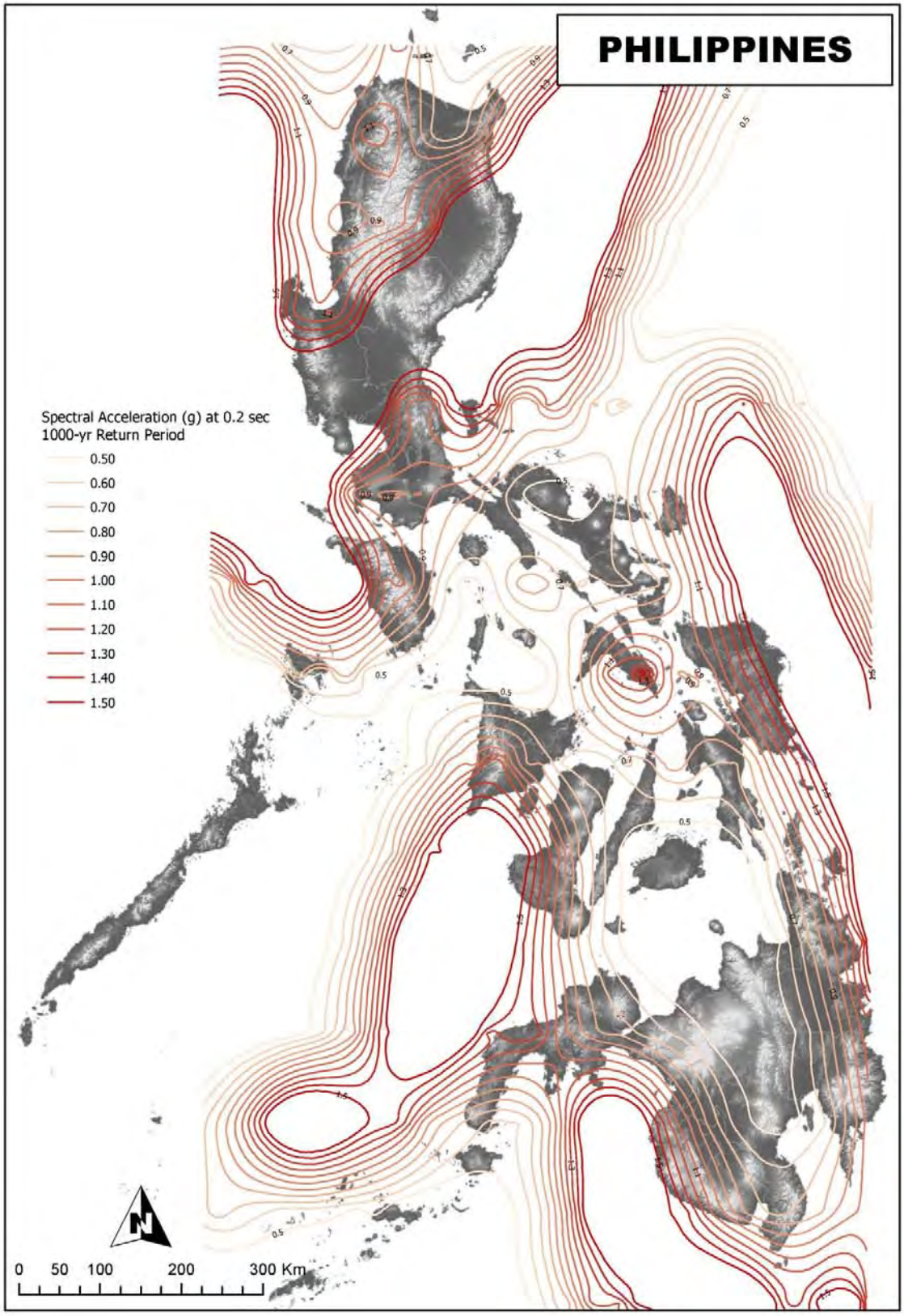


Figure 9.3-5 S_a at 0.2 sec for 1,000-year Return Period

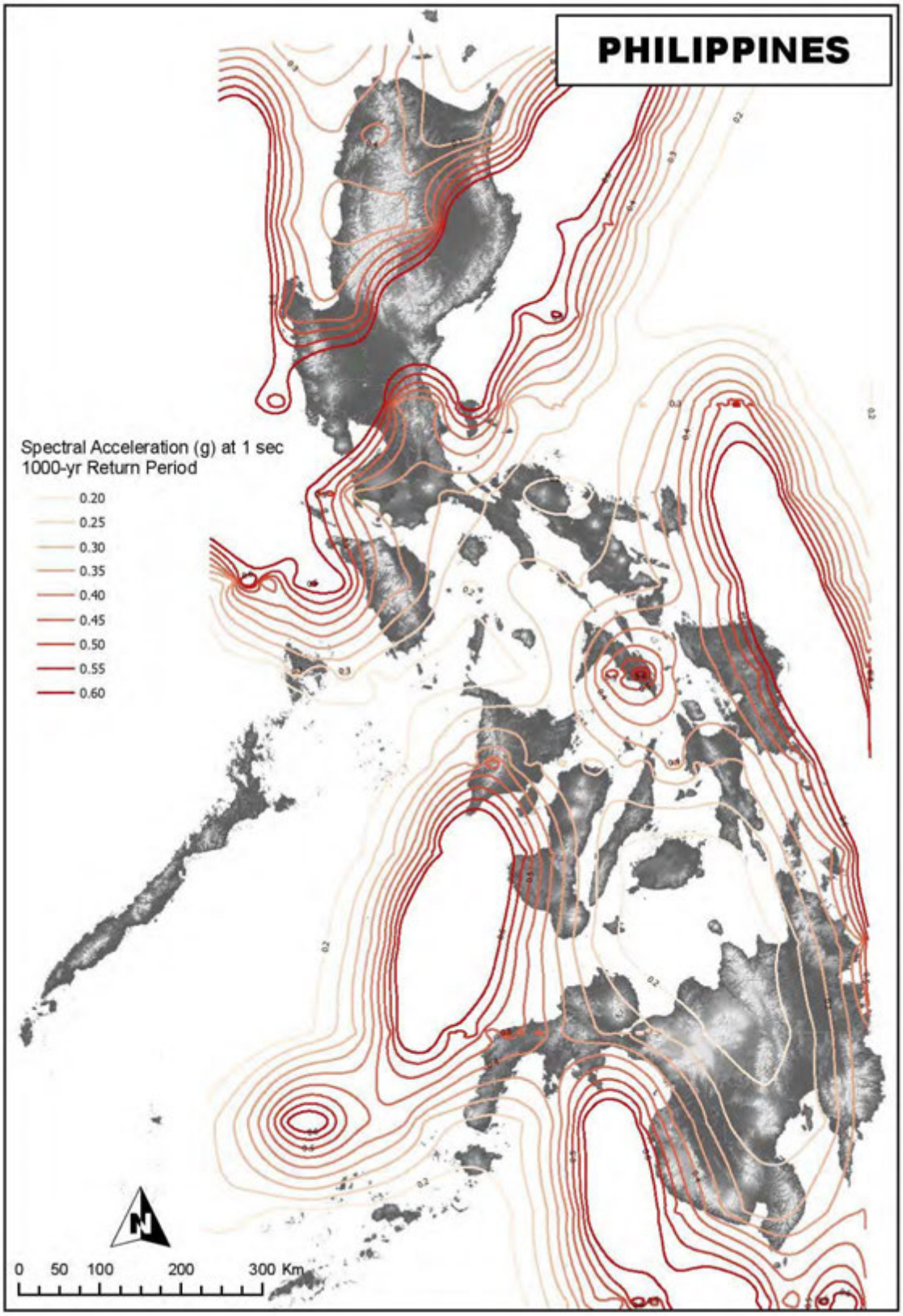


Figure 9.3-6 S_a at 1.0 sec for 1,000-year Return Period

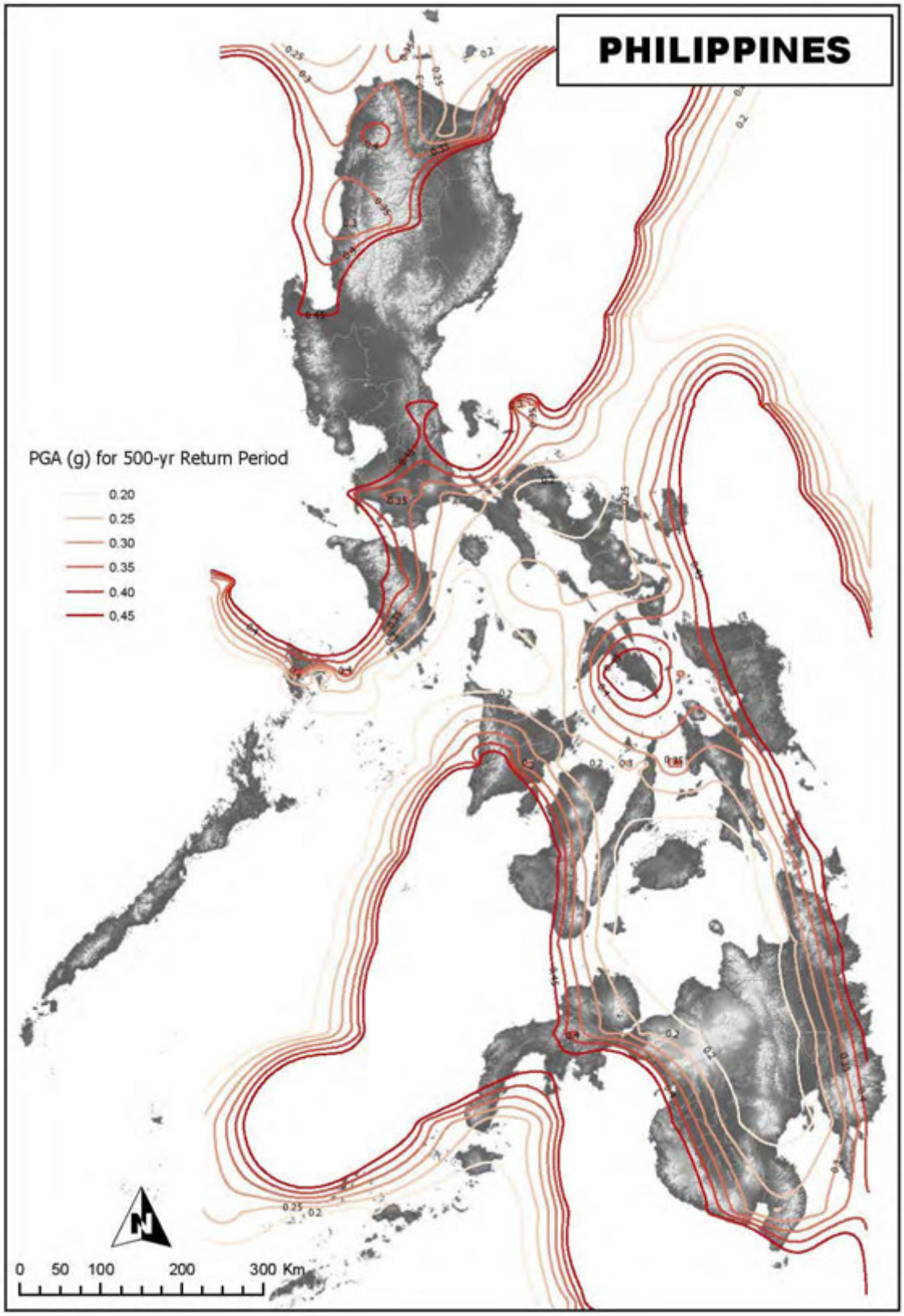


Figure 9.3-7 PGA for 500-year Return Period

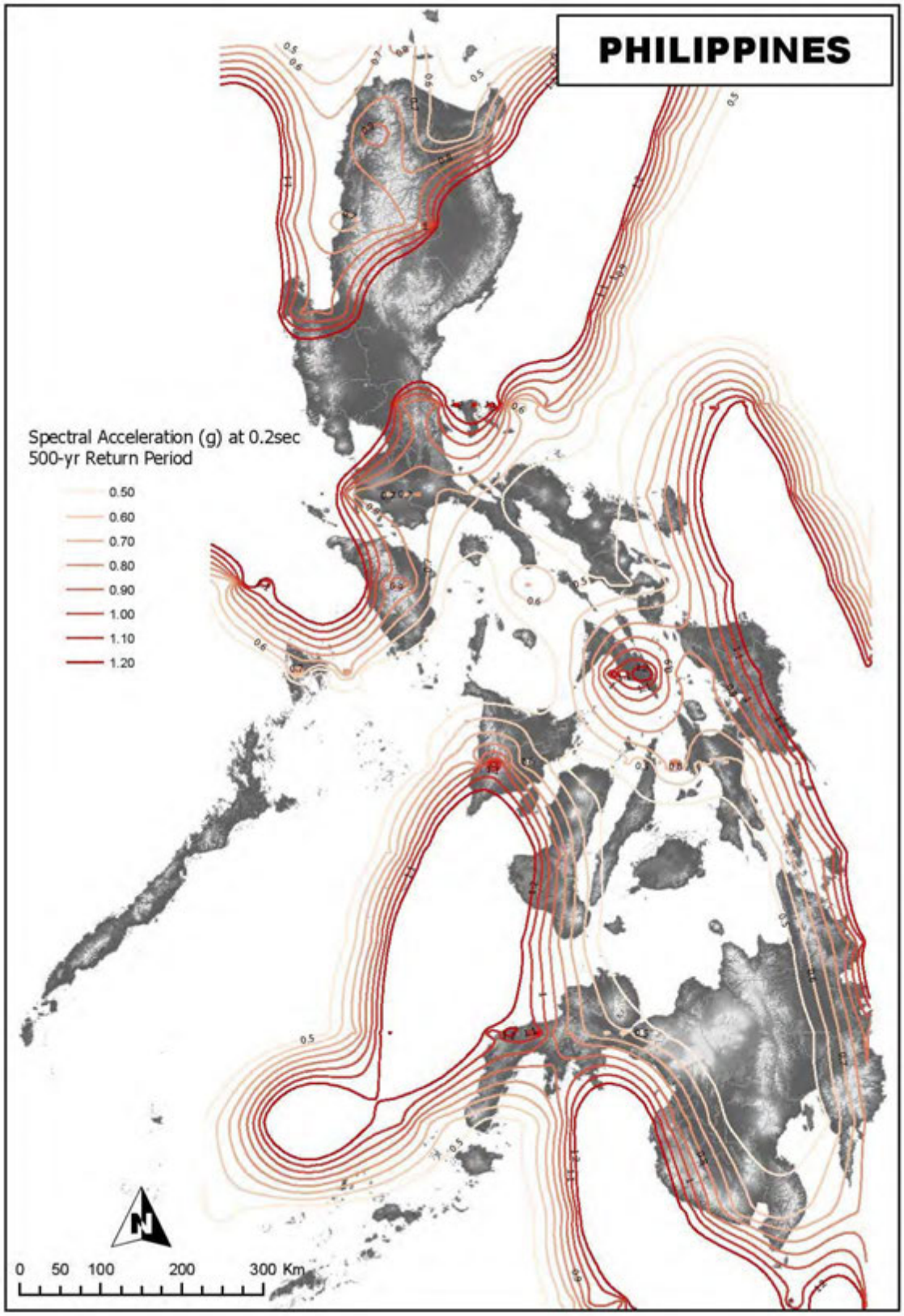


Figure 9.3-8 S_a at 0.2 sec for 500-year Return Period

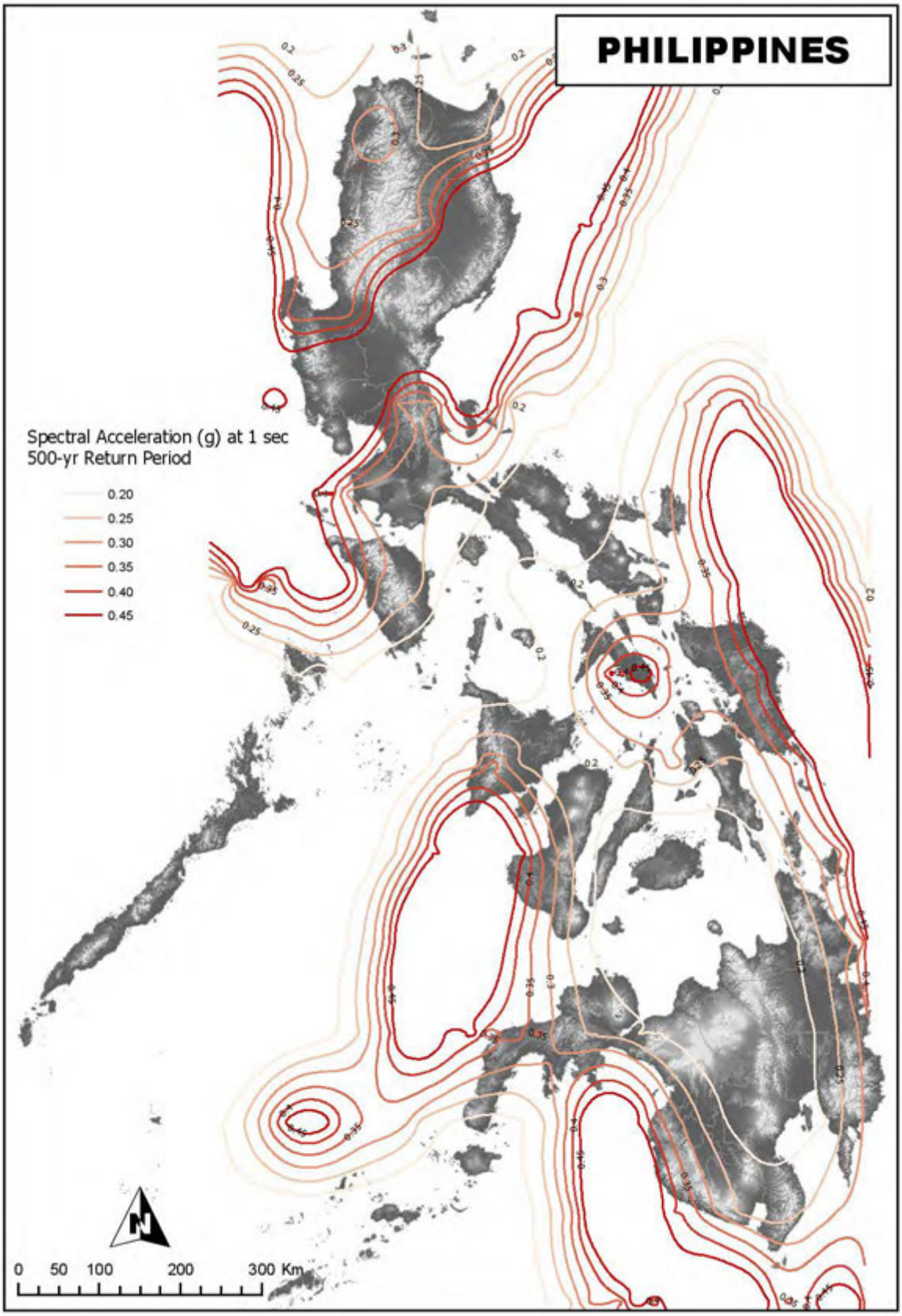


Figure 9.3-9 S_a at 1.0 sec for 500-year Return Period

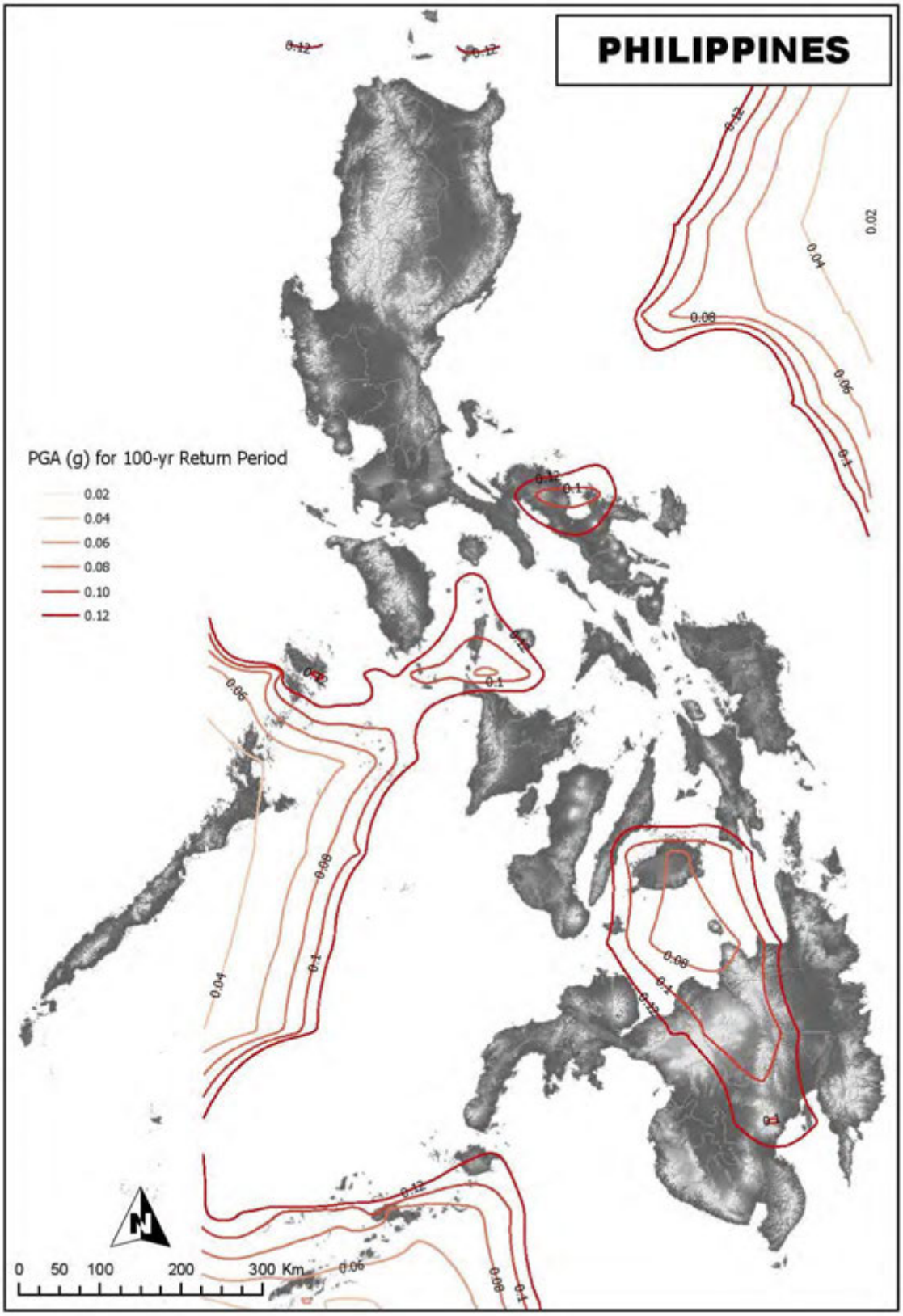


Figure 9.3-10 PGA for 100-year Return Period

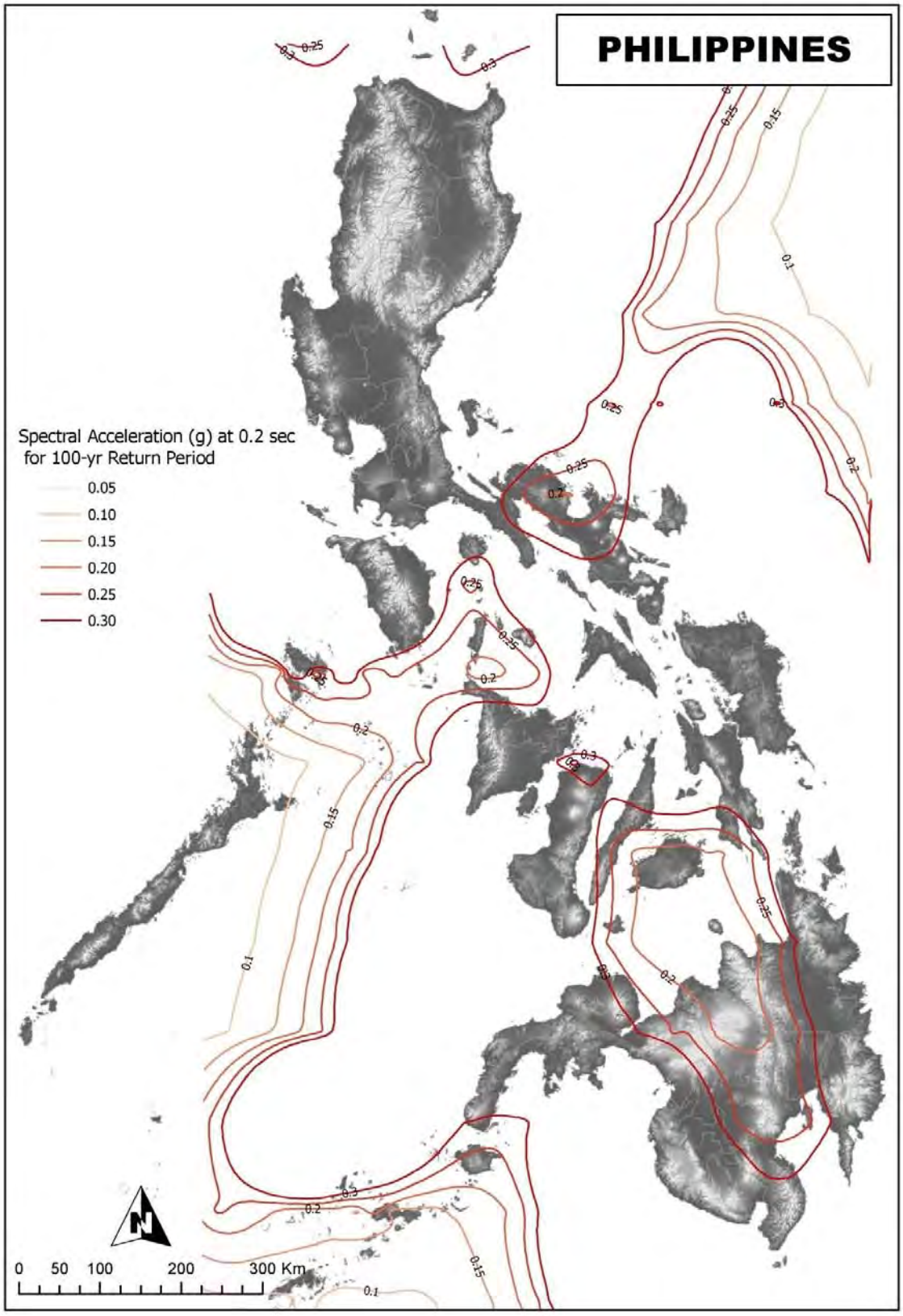


Figure 9.3-11 S_a at 0.2 sec for 100-year Return Period

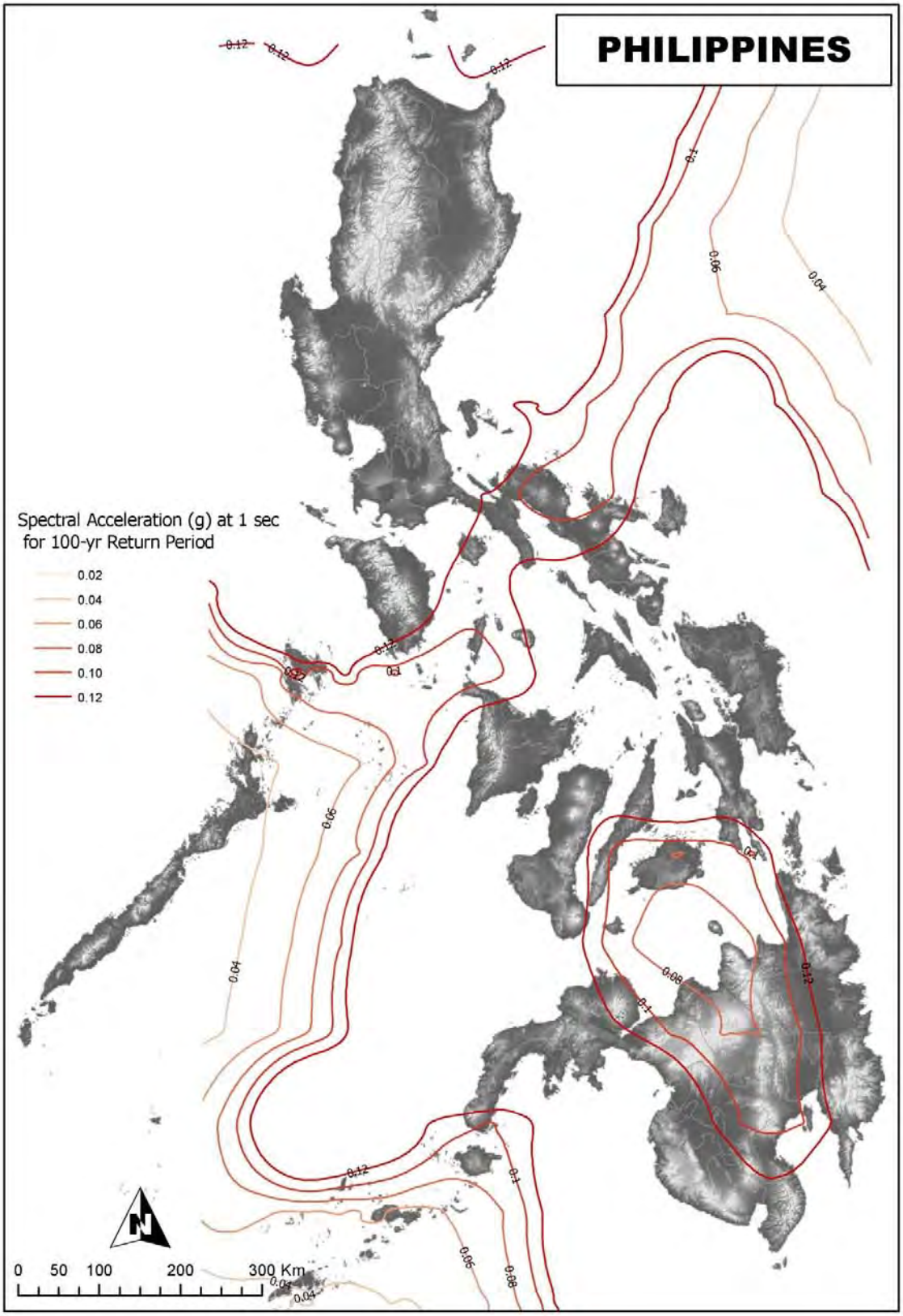


Figure 9.3-12 S_a at 1.0 sec for 100-year Return Period

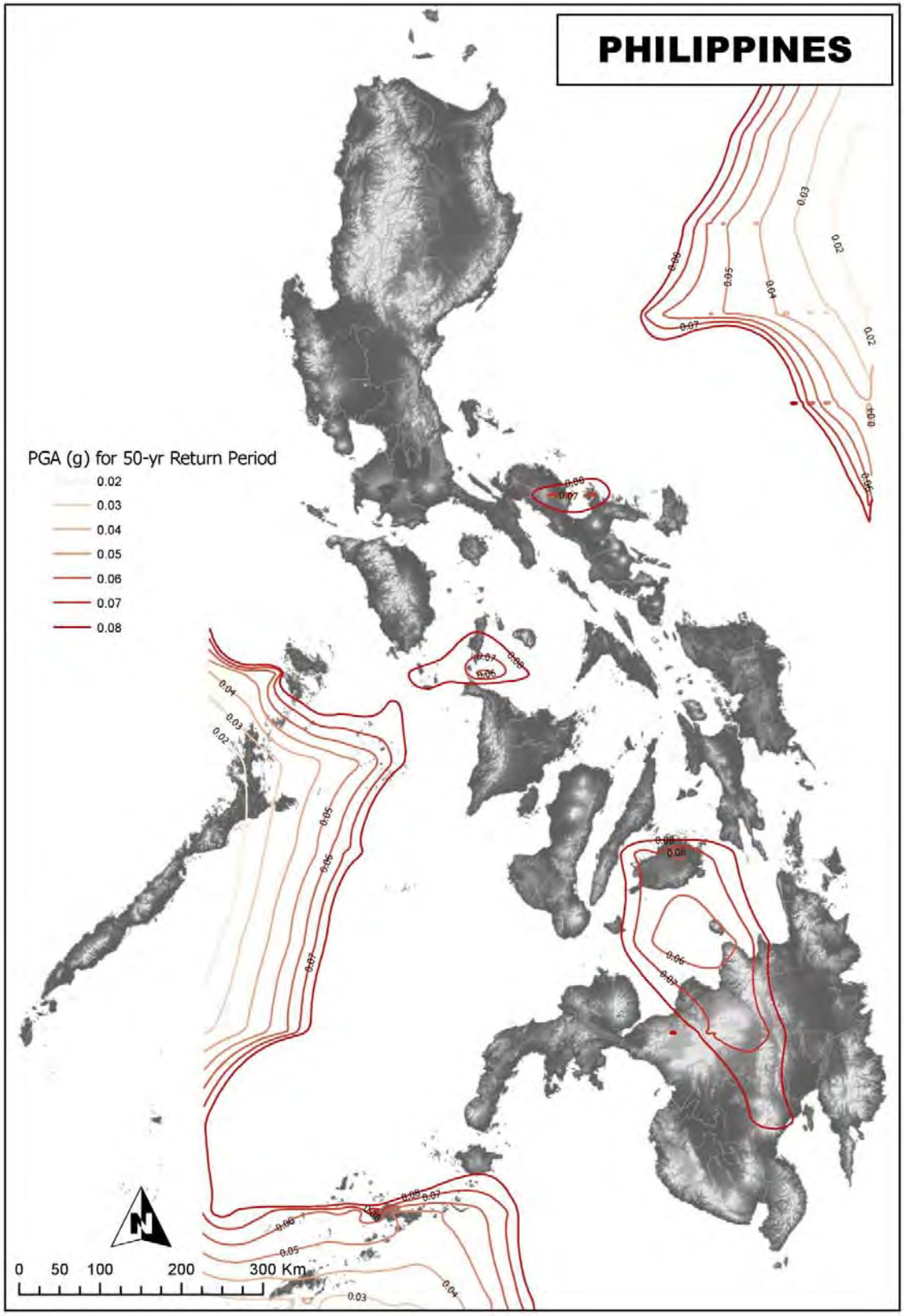


Figure 9.3-13 PGA for 50-year Return Period

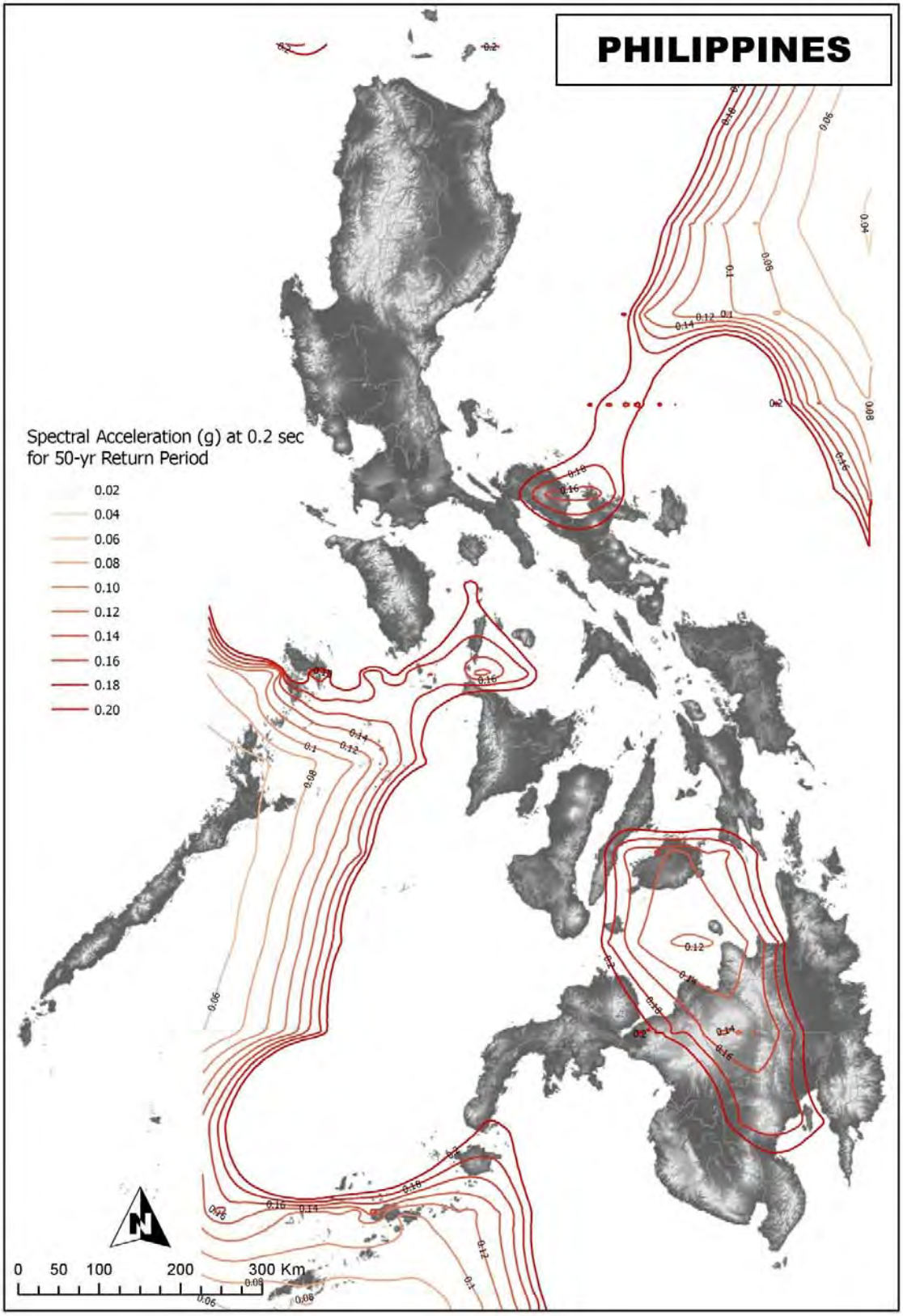


Figure 9.3-14 S_a at 0.2 sec for 50-year Return Period

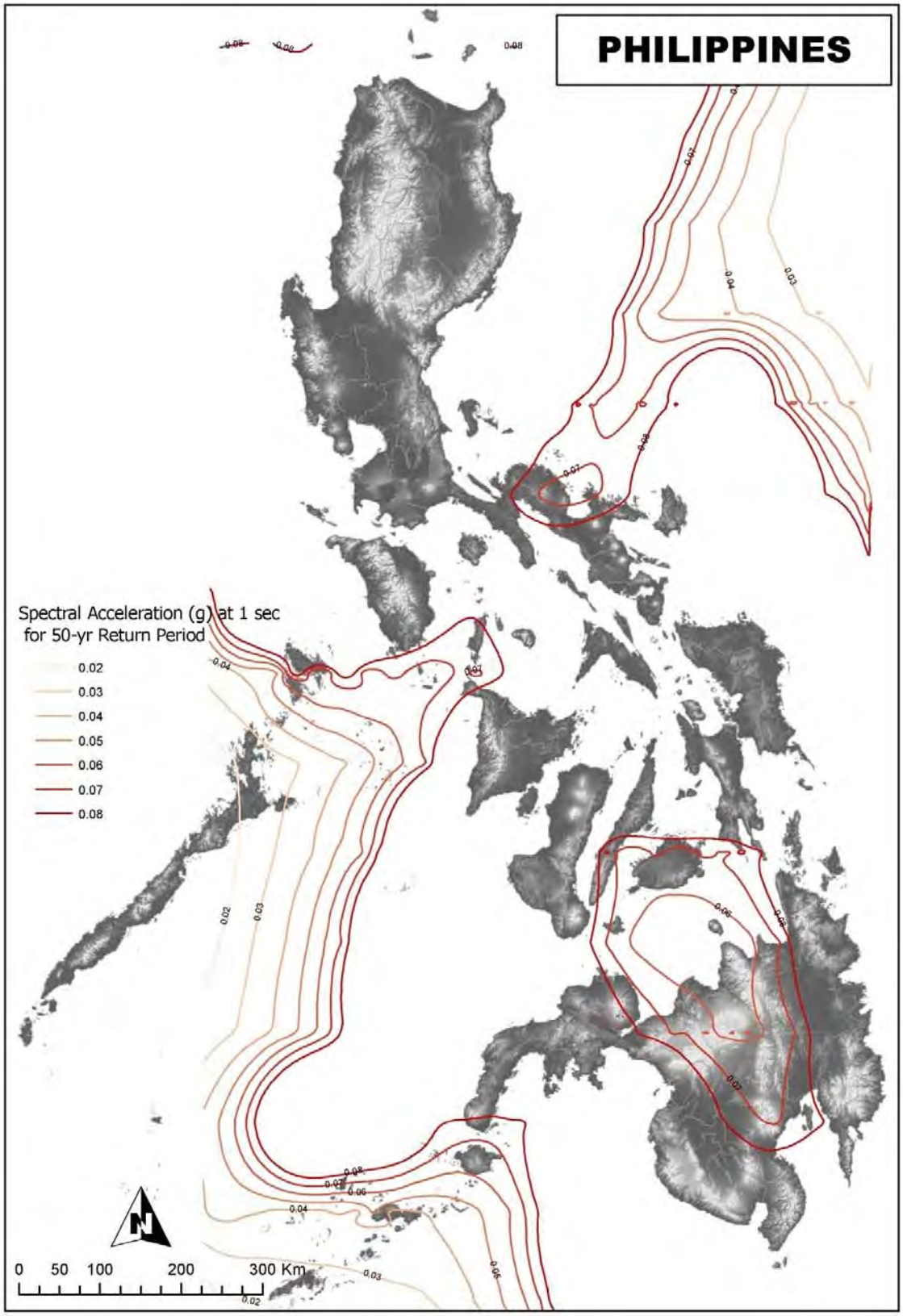


Figure 9.3-15 S_a at 1.0 sec for 50-year Return Period

Lastly, seismic contour map for PGA corresponding to 10% probability of exceedance in 50 years (equivalent to 500-year return period) from the 1994 Phivolcs-USGS study (Thenhaus et al, 1994) as

shown in Figure 9.3-16 is compared with similar results obtained in the present study as shown in Figure 9.3-17.

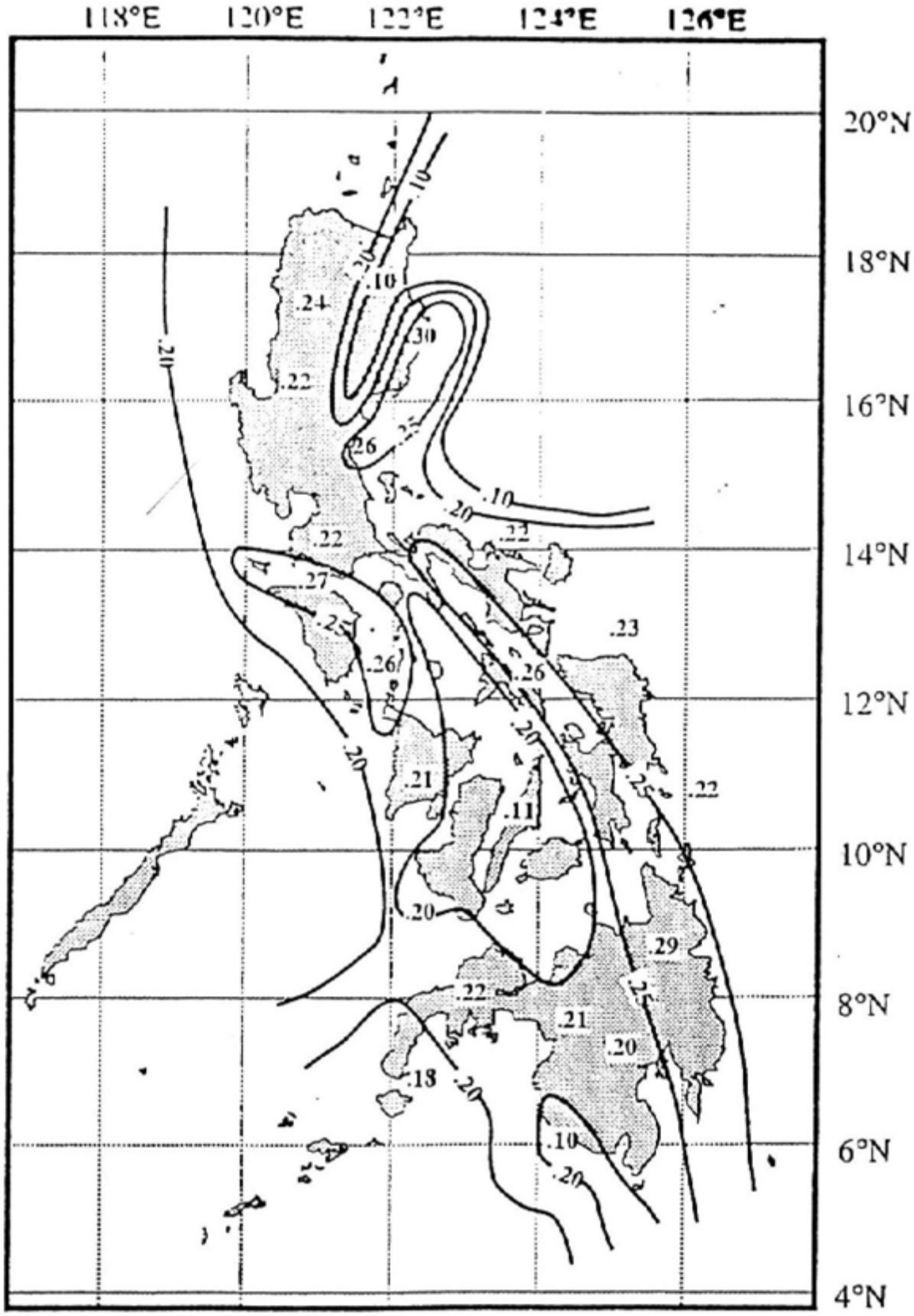


Figure 9.3-16 PGA for 500-year Return Period 1/2 (Thanhaus et al 1994)

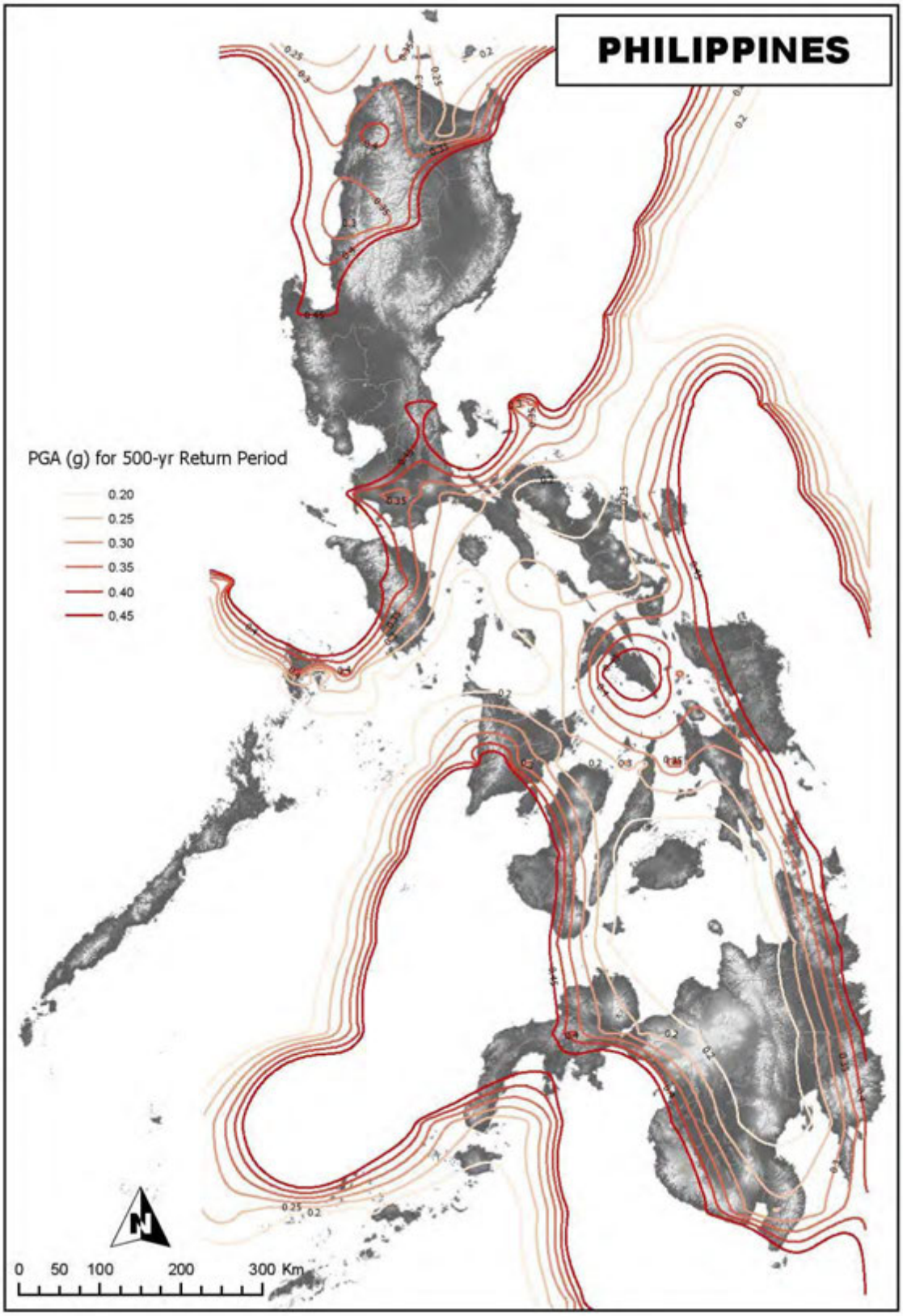


Figure 9.3-17 PGA for 500-year Return Period 2/2 (Present Study)

9.4 Site Effects

Local site effect can have strong influence on the earthquake motion at the ground surface. Generally, accelerations at the surface of soft soil deposits are larger than those on rock sites at low acceleration levels; and somewhat less at higher acceleration levels. And more importantly, subsurface soil conditions have a significant effect on the spectral shape.

The maps of spectral acceleration parameters are generated on ground conditions equivalent to AASHTO site class B (equivalent to Vs30 = 760 m/s which corresponds to the site class B/C boundary). The two-coefficient approach (one factor for short-period modification; and another factor for long-period modification) allows the incorporation of local soil condition effect on the ground motion response at the surface.

In the BSDS, 3 sets of generalized site modification factors for the 3 ground types and shaking intensity are tabulated to correct the mapped spectral acceleration values.

9.5 Assumptions and Limitations

Probabilistic seismic hazard analysis (PSHA) provides a framework in which uncertainties in the size, location, and rate of recurrence of earthquakes and in the variation of ground motion characteristics with earthquake size and location can be identified, quantified, and combined in a rational manner. Probabilistic seismic hazard analysis may be carried out with whatever little data are available; and updated when new information becomes available.

Seismic source characterizations are relatively uncertain owing to a lack of information characterizing the sources of seismic hazard, particularly the many faults that might be active. At present, there is a general lack of paleoseismic data for active faults which is important for characteristic earth-quake modeling. Phivolcs has continuing works on identifying active faults and estimating their slip rates, likely magnitudes, and recurrence rates.

A big source of uncertainties comes from estimation of earthquake ground motion. Selection of ground motion prediction models in this study relied on those modeling tectonically analogous regions of the US and Japan. Further study should include their applicability to Philippine use.

1. N. A. Abrahamson (2000). “State of the practice of seismic hazard evaluation.” *Proceedings of GeoEng 2000*, Melbourne, Australia, 19–24 November, Vol. 1, pp.659-685.

2. R. Dobry and S. Iai (2000). “Recent developments in the understanding of earthquake site response and associated seismic code implementation.” *Proc. GeoEng2000, An International Conference on Geotechnical & Geological Engineering*, Melbourne, Australia, 19–24 November, pp.186-129.

3. L. Reiter (1990). *Earthquake Hazard Analysis: Issues and Insights*. Columbia University Press.

4. P. C. Thenhaus, S. L. Hanson, S. Algermissen, B. C. Bautista, M. L. P. Bautista, B. Punong-bayan, A. R. Rasdas, J.-J. Nillos, and R. S. Punongbayan (1994). “*Estimates of the regional ground-motion hazard in the philippines*.” In Natural Disaster Mitigation in the Philippines, Proc. National Conf. on Natural Disaster Mitigation, DOST-PHIVOLCS, pages 45-60.

5. P. C. Thenhaus and K. W. Campbell (2003). “Seismic Hazard Analysis.”chapter 8 in *Earth-quake Engineering Handbook* (edited by W.-F. Chen and C. Scawthorne), CRC Press.

CHAPTER 10 OUTLINE OF DRAFT BRIDGE SEISMIC DESIGN SPECIFICATIONS (BSDS), MANUAL AND DESIGN EXAMPLES

10.1 Development of the Draft Bridge Seismic Design Specifications (BSDS)

10.1.1 Background

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 these guidelines are prepared in the early 1980s, the seismic design requirements and procedures are deficient and do not represent 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), as recommended in D.O. 75, is to refer to the AASHTO Standard Specifications for Highway Bridges (17th Edition, 2002) as the design specifications with minor revisions to suit local conditions. Design for earthquake forces is based on Division I-A (Seismic Design) of these Specifications 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 given 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.

Due to the urgent need to improve the seismic design guidelines in the wake of recent large earthquakes in the Philippines, the Japan International Cooperation Agency (JICA) undertook the project "*Study on Improvement of Bridges Through Disaster Mitigating Measures for Large Scale Earthquakes*" which is aimed at enhancing bridge performance under large earthquakes, including 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.

The key features of the bridge seismic design specifications cover:

- Establishment of bridge operational classification and seismic performance requirement of bridges in the Philippines,
- Localization of the Philippine design seismic ground acceleration map, including the corresponding seismic design response spectra,
- Adoption of three (3) ground types for site classification and site effects in seismic design,
- Adoption of the *AASHTO LRFD Bridge Design Specifications, 2012 edition* (force-based, R-factor method) as the base specifications,
- 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).

10.1.2 Need for Revision of Current Bridge Seismic Specifications

After a thorough review of the existing design guidelines and current design practice for seismic bridge design under the DPWH the development of the new BSDS is justified by the following needs:

(1) Design Seismic Performance Requirements

The seismic performance levels shall be defined as the desired performance behavior of bridges in the event of an earthquake considering bridge type, importance classification and earthquake ground motion. However, the DPWH design philosophy stating that "bridges to resist small to moderate earthquakes in the elastic range without significant damage" and "in case of large earthquakes, bridges may suffer damage but should not cause collapse of all or any of its part", does not define explicitly the seismic performance requirements in terms of safety, serviceability and repairability. Thus, there is a need to verify seismic performance design corresponding to the level of design ground motion at the desired seismic performance.

(2) Importance Classification (IC)

Although Essential bridges (IC=I) and Other bridges (IC=II) are basically the two importance classification used to determine the seismic performance categories of bridges in the DPWH, there is no specific definition as to which bridge falls under each categories. Moreover, the latest AASHTO classifies bridges as to Critical, Essential and Others. There is a need to establish a more definitive importance classification of bridges corresponding to road class and function, bridge function and structural characteristics and socio-economic functions.

(3) Site Classification and Soil Profile Types

The DPWH seismic design guidelines still refer to the site classification and soil types defined in the 1996 AASHTO Specifications, covering four soil profile types. However, such definitions of the soil profile types are inherent with the soil site characteristics in the U.S.A. and do not necessarily reflect the ground conditions in the Philippines. A more realistic soil profile types based on the actual local conditions is thus necessary, indicating the need to revise the existing seismic design specifications.

On the other hand, JRA classifies three ground types which are more closely related to the Philippines soil characteristics. Classification and characteristics of JRA ground types will be used as the base of the revised design specifications. Moreover, the relationships between N-values, shear wave velocity and characteristic values of ground surface will be established.

(4) Peak Ground Acceleration (PGA) Coefficient

The ground acceleration coefficient used in seismic design of bridges is based on the 2-zone seismic map of the Philippines provided in the *National Structural Code of the Philippines (NSCP) Vol. II (Bridges)*, 1997. However, the basis of the map is not clear considering the active faults and ocean trenches running through almost the entire Philippine archipelago. Moreover, the map does not indicate the frequency of the design earthquake.

The previous AASHTO seismic design specifies design earthquake with almost 500-year return period. However, the present AASHTO Specifications stipulates a 1,000-year return period. In this regard, it would be necessary to establish a more realistic ground acceleration coefficient map for the entire Philippines considering the presence of active faults and ocean trenches and the probability of occurrence of the design ground motion.

Developed peak ground acceleration coefficient, under this project, for the 50-year, 100-year, 500-year and the 1,000 year return period earthquakes were compared and with 100-year and 1,000-year return periods proposed for the Level 1 and Level 2 earthquake ground motions.

(5) Design Response Acceleration Spectra

Basically, the DPWH specifies a seismic response coefficient or the response acceleration spectra based on the AASHTO response acceleration spectra for 5% damping for various soil profile to define the earthquake load to be used in the elastic analysis for seismic effects. In the same reason previously stated, these spectra and coefficient do not necessarily reflect the earthquake ground motion characteristics in the Philippines. In which case, it is necessary to establish localized seismic design response acceleration spectra reflecting the site characteristics and probable ground motion of the design earthquake in the Philippines. The proposed BSDS will adopt the methodology given in the AASHTO LRFD Specifications.

(6) Ground Liquefaction

Although AASHTO specifies some provisions in assessing ground liquefaction potential in the Philippines, it is not clear as to how to apply such for seismic design of bridges. The JRA Specifications presents a more specific guideline on how to consider ground liquefaction for bridge seismic design – such provision are proposed to be adopted to the Philippine seismic design of bridges.

(7) LRFD Design Philosophy

The present design of bridges by DPWH employs WSD and LFD design philosophies, based on the AASHTO WSD/LFD Specifications (the last AASHTO edition of which is in 2002, 17th Ed.). However, with the recent trends to provide a more systematic and rational approach to the selection of load factors and resistance factors using statistics and reliability theory, AASHTO shifted its design philosophy to load and resistance factor design (LRFD). This will increase the uniformity of the margin of safety and reliability of bridge structures and eliminate gaps and inconsistencies in the specifications related to variability of loads and resistances. Thus the need to adopt the LRFD philosophy for DPWH seismic design of bridges is indicated.

(8) Unseating/Fall-down Prevention System

Unseating and fall-down of bridge superstructures have been observed in recent major earthquakes in the Philippines. In this regard, the DPWH current practice for seismic design of bridges requires provisions for preventing the superstructure from unseating in the event of an unexpectedly large earthquake. However, details of the unseating/fall-down prevention system in the design guidelines need to be organized with additional supplemental provisions from JRA.

(9) Seismically Isolated Bridges

Although the use of seismic isolation devices for bridges is not common in the Philippines, bridges with short natural periods and those founded on hard soil layers may be suitable for seismic isolation to reduce the forces going into the substructures. Provisions for seismically isolated bridges based on JRA is introduced in the proposed seismic design specifications.

(10) Foundation Design

The use of JRA method in foundation design including the determination of bearing capacities of spread footings and piles is widely used in the Philippines. Incorporation of the JRA method for foundation design to the revised seismic design specifications is considered.

Calculation and use of soil spring constants (both static and dynamic) between the foundation structure and the ground are explicitly stated in the JRA specifications, which can easily be applied to model the foundation. Such spring constants are not stated in the current DWPH specifications.

10.1.3 Policy on the Development of Bridge Seismic Design Specifications (BSDS)

The development of the BSDS follows certain philosophy as given in the following:

(1) Purpose of Specifications

The Specifications are intended to:

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

The BSDS is intended to guide the DPWH engineers and the design professionals for the minimum requirements in the design of 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 bridges. Moreover, such technologies which are not covered in the BSDS shall be subject to the approval of the DPWH.

(2) Scope of Specifications

- The BSDS covers eight (8) sections as follows:

  Section 1 : Introduction
  Section 2 : Definitions and Notations
  Section 3 : General Requirements
  Section 4 : Analysis Requirements
  Section 5 : Design Requirements
  Section 6 : Effects of Seismically Unstable Grounds
  Section 7 : Requirements for Unseating Prevention System
  Section 8 : Requirements for Seismically Isolated Bridges

- The scope of the BSDS covers mainly seismic design of bridges under the "Extreme Event Limit State for Earthquake Loading (Extreme Event 1)" following the design concept and philosophy of the *AASHTO LRFD Bridge Design Specifications* (2012 or later). However, the provisions for other limit states shall be referred to the AASHTO LRFD Bridge Design Specifications or the DPWH Specifications which is under development in a separate project.

- The BSDS, following the AASHTO LRFD design methodology, adopted the force-based R-factor design approach to account for column ductility. The R-factors (referred to in the BSDS as the response modification factors or force-reduction factors) accounts for the strength and ductility requirements for the seismic performance levels of corresponding bridges under the "critical, essential and others" operational classifications. The R-factors are then specified to determine the inelastic deformation demands on the bridge members when the design earthquake occurs.

- The applicability of the BSDS with regards to the types of bridges covers conventional structural form and construction method with slab, beam, box girder and truss superstructure with pier and pile bent substructures founded on shallow or piled footings or shafts. However, appropriate provisions of the BSDS may be adopted for non-conventional bridges and other types of construction (e.g. suspension bridges, cable stayed bridges, arch type bridges, and movable bridges) or foundations, provided prior approval by the DPWH is obtained.

- The provisions given in the BSDS is taken to be 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 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.

- Since the BSDS is intended to cover only the extreme event limit under earthquake, other provisions not contained in the BSDS shall refer 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 – Substructure*s and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

(3) Organization of Specifications

The Specifications are written to provide guidance to engineers in the design of bridges under earthquake loading for the extreme event limit state. As such, the BSDS is organized as:

- The main specifications for extreme event under earthquake loading are given in the box for engineers to comply with the design requirements.
- For other analysis and design requirements, the designer shall comply with the *AASHTO LRFD Bridge Design Specifications*, which is the base specification. However, since the DPWH is preparing its version of the LRFD design specifications, the BSDS shall form part of the DPWH LRFD specifications once it is completed.
- In order to deepen the understanding by bridge engineers and practitioners on the use and application of the BSDS, Commentaries are prepared after each Article of the Specifications. This will guide the engineers on the proper use and how to apply the BSDS to the design of bridges.

  The Commentaries contain some background information, principles applied, detailed procedures and recommendations on interpreting the Specifications. By guiding the users of the Specifications, the Commentaries will also serve as an accompanying Manual for the BSDS. It is hoped that the Commentaries will encourage the practicing engineers and promote utilization of the BSDS.

  However, although the Commentaries provide guidance to practicing engineering in applying the provisions of the BSDS in seismic design, the engineers may also use other acceptable design procedures and references (acceptable to DPWH) that can provide more efficient design of bridges.
- In addition to the above, technical training, seminar and workshop on bridge seismic design technology are held during the development stage of the BSDS for the DPWH engineers and private practitioners so as to improve their technical knowledge and skills in seismic design.
- A training/workshop will be held for the DPWH bridge design engineers and bridge design practitioners on the use of the BSDS.

10.2 Outline of the Draft Bridge Seismic Design Specifications (BSDS)

This section gives brief summary outline of the proposed bridge seismic design specifications.

10.2.1 Section 1 : Introduction

Background. The background on the development of the BSDS is presented in this Section summarizing the key features of the specifications to include the seismic performance requirements and bridge operational classification, localization of the design response spectra thru acceleration coefficient maps generated by probabilistic seismic hazard analysis, three ground types for site class, effects of soil liquefaction and lateral spreading, unseating prevention system and seismic isolation system. The guide specification is prepared to cover the seismic design of bridges in the Philippines that will guide the DPWH and the civil engineering professionals for the minimum requirements for seismic design.

Purpose. The BSDS is intended to provide guidance to DPWH and professional engineers in the seismic design of bridges that sets the minimum requirements for seismic design integrity and safety under a large earthquake. Further, the BSDS establishes the design provisions that will minimize damage and guarantee the required seismic performance from large earthquake.

Scope. The BSDS covers mainly the design of conventional bridges under extreme event limit state for earthquake loading following the AASHTO philosophy for LRFD methodology using force-based design approach.

Seismic Design Philosophy. The philosophy of load and resistance factor design is adopted using the force-based and capacity design approach to design members under earthquake loading. Reference is made to the DPWH Department Order No. 75 for the design concept.

Two levels of earthquake ground motions are considered:

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

The load combination specified is based on the AASHTO Extreme Event Load Combination I, combining permanent loads and transient loads with earthquake loading. Live load effects during earthquake shall be considered by applying half of the design live load in the superstructure.

The seismic design procedure flowchart is presented in Figure 10.2.1-1.

An Appendix highlighting the earthquake resisting systems and elements taken from the AASHTO Guide Specifications for LRFD Seismic Bridge Design is given as a reference to this section.

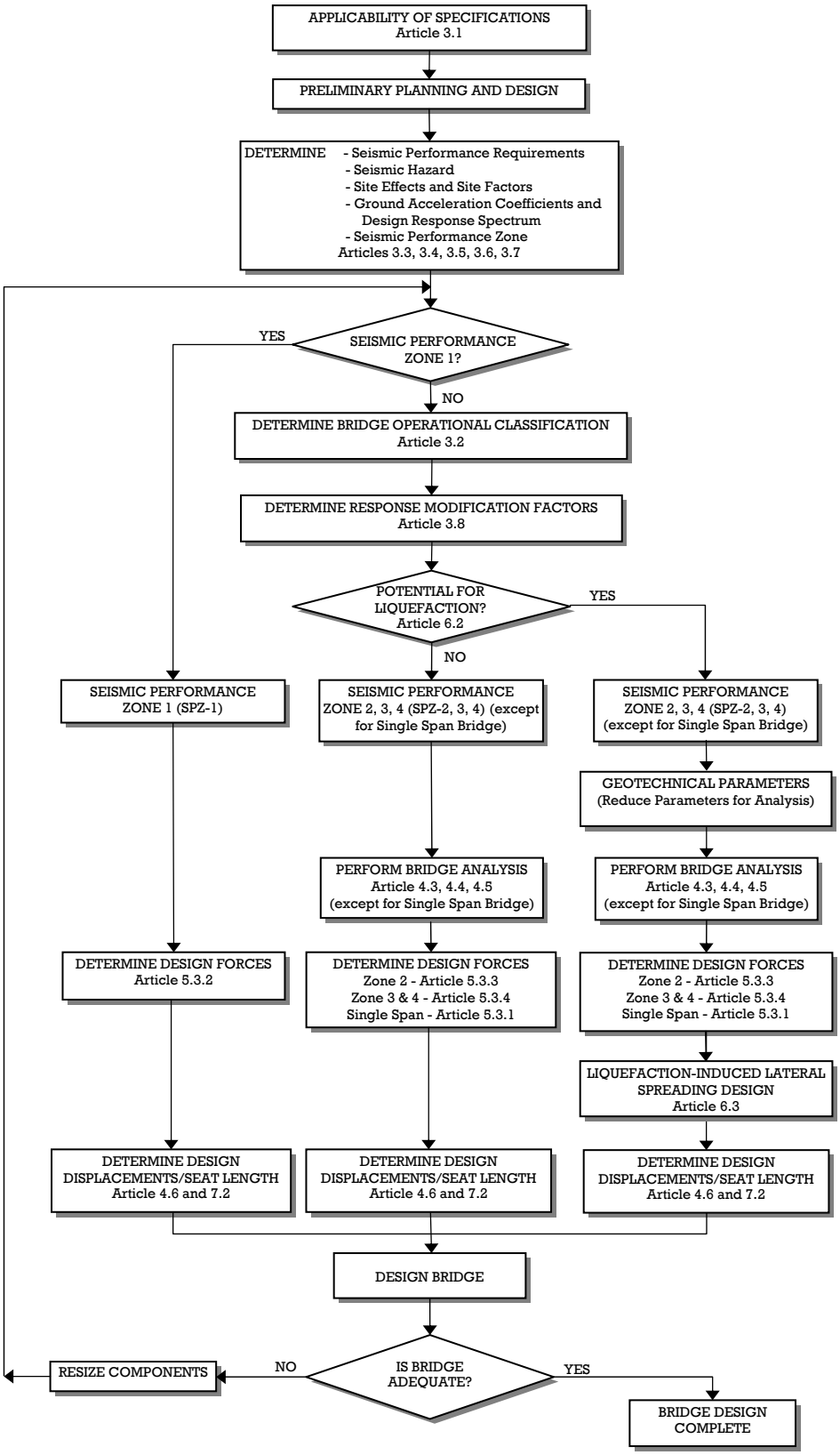


Figure 10.2.1-1 Seismic Design Procedure Flow Chart

10.2.2 Section 2 : Definitions and Notations

The definitions of common terminologies and the symbol and abbreviation notations are presented in this section.

10.2.3 Section 3 : General Requirements

The section on General Requirements specifies the following:

(1) *Applicability of Specifications:* The BSDS is taken to apply to the design and construction of conventional bridges to resist the effects of earthquake motions. For non-conventional or other bridge types, the BSDS may be applied with additional design requirements as required by the DPWH.

(2) *Bridge Operational Classification.* Table 10.2.3-1 presents the operation classification of DPWH bridges.

Table 10.2.3-1 Operational Classification of Bridges

Operational Classification (OC)	Serviceability Performance	Description
OC-I (Critical Bridges)	• Bridges that must remain open to all traffic after the 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 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 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

(3) *Seismic Performance.* Bridges are expected to perform under three levels of performance as summarized in Table 10.2.3-2 and illustrated in Figure 10.2.3-2 below.

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.

Table 10.2.3-2 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)	**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)*

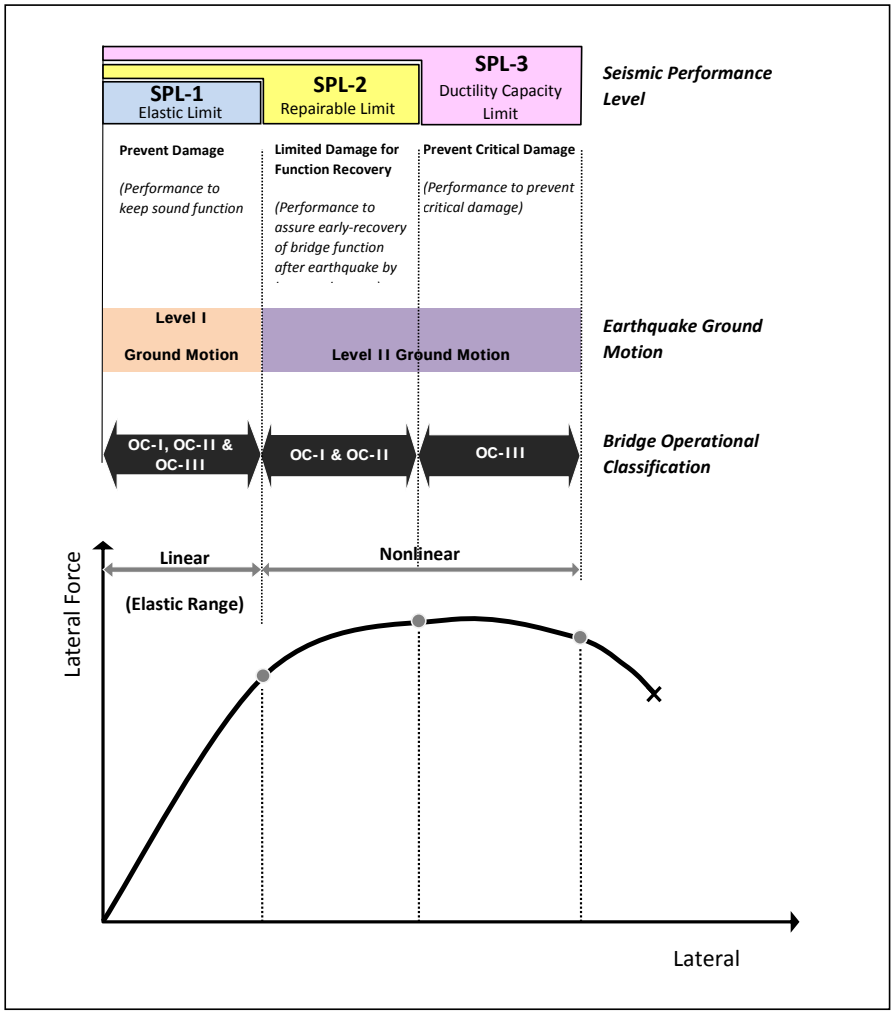
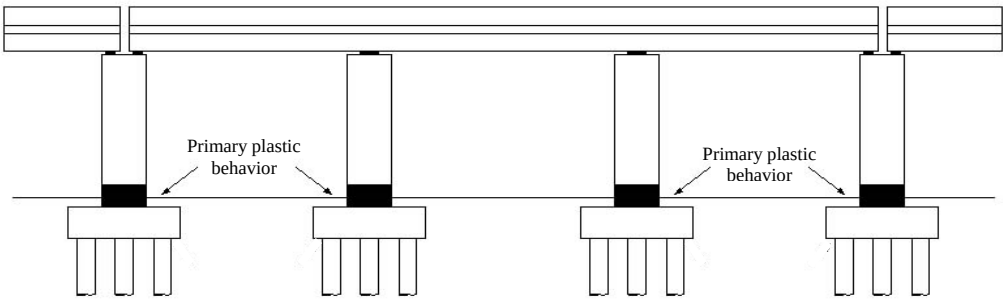
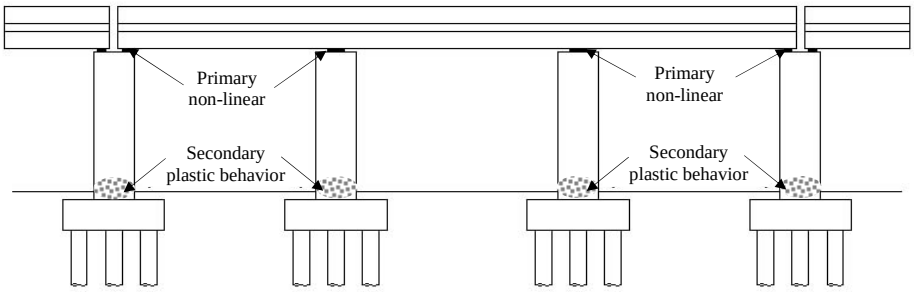


Figure 10.2.3-1 Relationship between Lateral Load-Displacement Curve, Seismic Performance Level, Earthquake Ground Motion and Operational Classification

Typical forms of structural plasticity or non-linearity is shown in Figure 10.2.3-2 below.

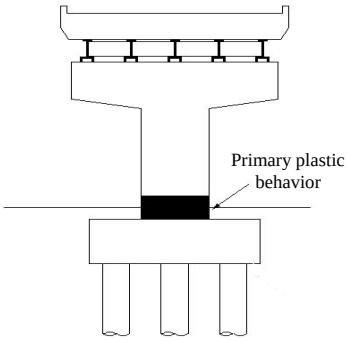


(c) Single Column Pier with Plastic Behavior
(longitudinal direction)

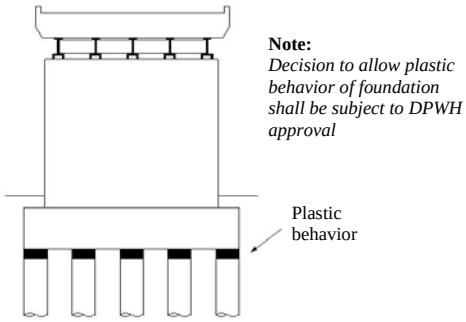


Seismically-Isolated

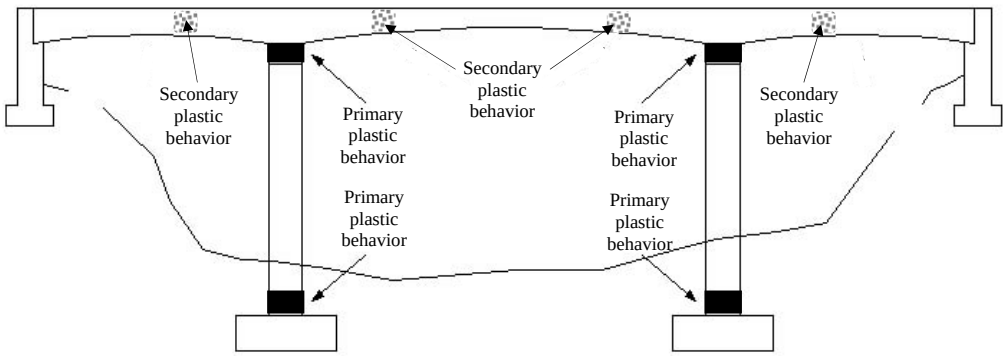
(a) Seismic Isolation Bearings with Consideration of Non-linearity



(b) 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 10.2.3-2 Combination Examples of Members with Consideration of Plasticity or Non-Linearity

The descriptions of member behavior for Seismic Performance Levels 2 and 3 are shown in Tables 10.2.3-3 and 10.2.3-4, respectively.

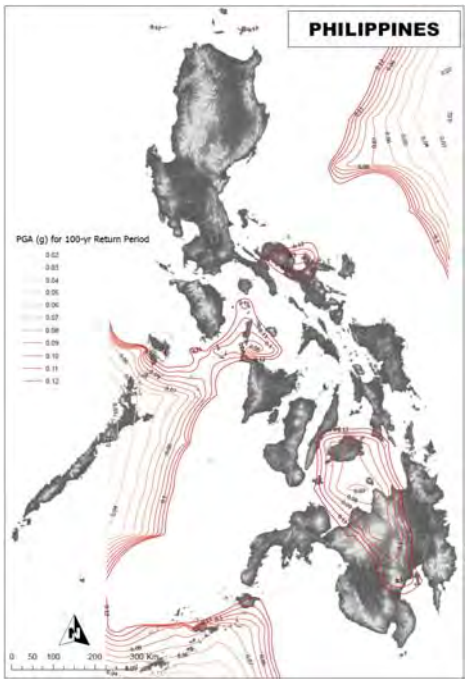
Table 10.2.3-3 Combination Examples of Members Considering Plasticity (Non-linearity) and Limit States of Each Members (For Seismic Performance Level 2)

Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers	Piers and Superstructures[1)]	Foundations[2)] (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

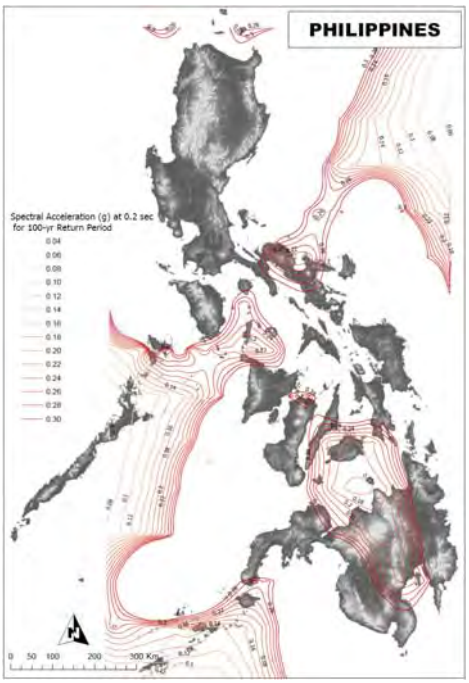
Table 10.2.3-4 Combination Examples of Members with Consideration of Plasticity (Non-linearity) and Limit States of Each Members (For Seismic Performance Level 3)

Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers	Piers and Superstructures	Foundations	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

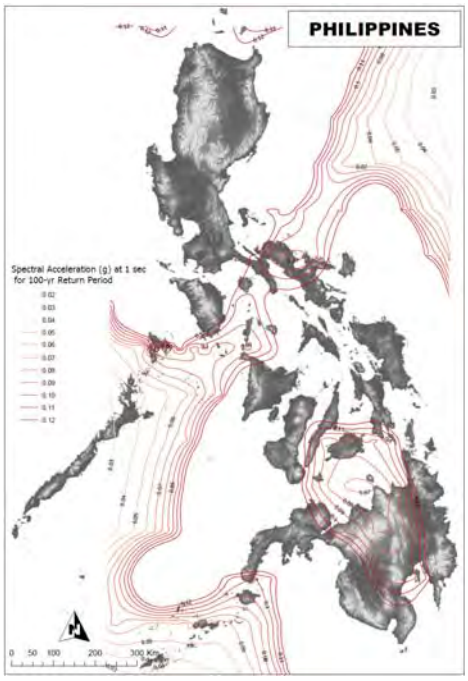
(4) *Seismic Hazard.* The seismic hazard at the bridge site will be characterized by the acceleration response spectrum for the site and the site factors for the relevant site class. Figure 10.2.3-3 shows the seismic hazard maps for a 100-year return earthquake while Figure 10.2.3-4 shows the seismic hazard maps for a 1,000-year return earthquake. However, these maps are derived based on a basement rock corresponding to AASHTO Type "B" soil profiles. The effects of ground amplification due to the existing ground type shall be adjusted using site factors.



(a) PGA

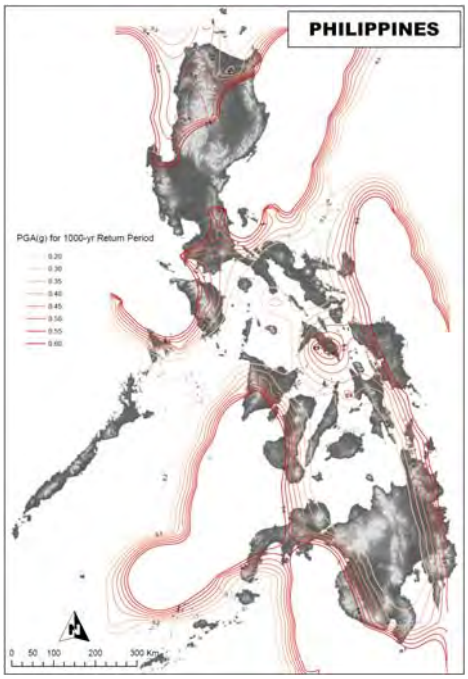


(b) 0.2-sec Response Spectral Acceleration

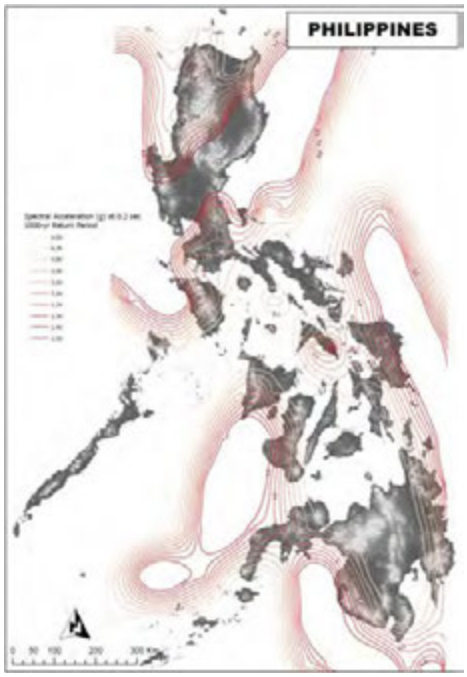


(c) 1-sec Response Spectral Acceleration

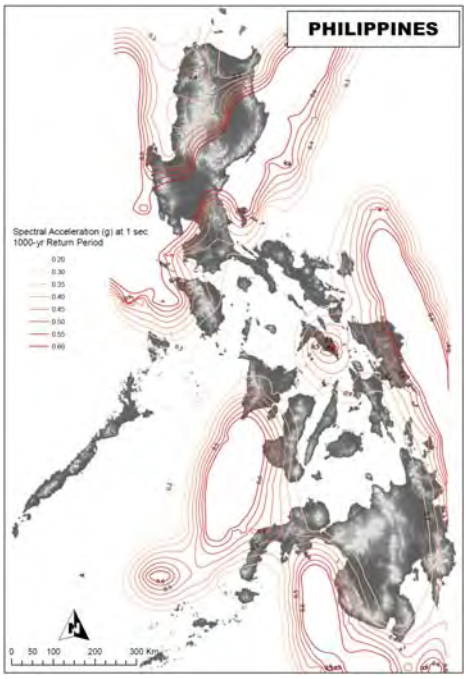
Figure 10.2.3-3 Seismic Hazard Maps for a 100-year Return Earthquake



(a) PGA



(b) 0.2-sec Response Spectral Acceleration



(c) 1-sec Response Spectral Acceleration

Figure 10.2.3-4 Seismic Hazard Maps for a 1,000-year Return Earthquake

(5) *Ground Types.* Three ground types for seismic design are specified based on the characteristic ground values, T_G given in Table 10.2.3-5.

Table 10.2.3-5 Ground Types (Site Class) for Seismic Design

Ground Type	Characteristic Value of Ground, T_G (s)
Type I	$T_G < 0.2$
Type II	$0.2 \le T_G < 0.6$
Type III	$0.6 \le T_G$

(6) *Design Response Spectrum and Site Factors*. The design response spectrum, based on the AASHTO LRFD procedure, is a three-point 5% damped response spectra developed based on the seismic hazard maps of PGA and acceleration response coefficients as illustrated in Figure 10.2.3-5. Once the coefficients for PGA, 0.2-sec and 1-sec response acceleration are determined, site factors are applied to adjust for the effects of amplification due to the existing ground type.

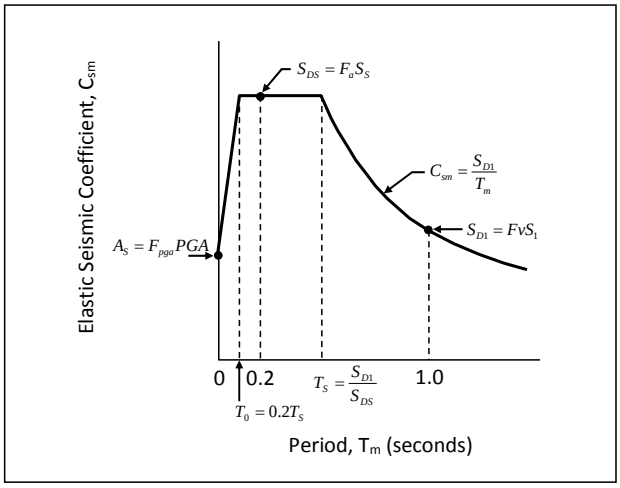


Figure 10.2.3-5 Design Response Spectrum

(7) *Response Modification Factors (Site Factors).* The force effects resulting from elastic analysis shall be divided by the appropriate response modification factor (*R*-factor) to determine the seismic design forces for substructures. Table 10.2.3-6 indicates the corresponding *R*-factors based on the bridge operational classification.

Table 10.2.3-6 Response Modification Factors for 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	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical piles only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

10.2.4 Section 4 : Analysis Requirements

This section describes the analysis requirements for seismic effects on bridges under the extreme event limit states.

(1) *Minimum Analysis Requirements:* Table 10.2.4-1 specifies the minimum analysis requirements for bridges under earthquake loading.

Table 10.2.4-1 Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	MM/TH
4		SM/UL	MM	MM	MM	MM/TH	MM/TH

where:

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

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

The boundary conditions shall represent the actual characteristics of support and continuity. The foundation conditions shall be modeled in such a manner to represent the soil properties underlying the bridge, the soil-pile interaction, and the elastic properties of piles. As an initial analysis model to determine the design forces for foundation, the piers shall be assumed to be fixed at the ground surface for seismic design.

The static and dynamic coefficients of subgrade reactions are given in this section to provide an equivalent model of the effects of soil-structure behavior during earthquake.

(3) *Dynamic Analysis Requirements:* The stiffness, mass and damping characteristics of the structure shall be realistically modeled to analyze the dynamic behavior of bridges.

In seismic analysis, nonlinear effects of which decrease stiffness, such as inelastic deformation and cracking, should be considered. Reinforced concrete columns and walls in Seismic Performance Zones 2, 3 and 4 should be analyzed using cracked section properties. For this purpose, a moment of inertia equal to one-half that of the uncracked section may be used.

(4) *Minimum Seat Length Requirements:* Adequate measures against unseating of superstructures shall be taken when the superstructure separates structurally from the substructure, and with large relative displacements. The seat width shall be consistent with the unseating prevention system (Section 7).

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

10.2.5 Section 5 : Design Requirements

This section describes the design requirements for seismic effects on bridges under the extreme event limit states.

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

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

However, when foundation and/or column connection forces are determined from plastic hinging of the columns, the resulting force effects may be determined without consideration of combined load cases given above.

(2) Calculation of Design Forces:

- For single-span bridges, the minimum design connection forces shall be the force effect taken from the product of the acceleration coefficient A_S and the tributary permanent load.
- Seismic Performance Zone 1 – the minimum design connection forces shall be 0.15 times the vertical reaction when A_S is less than 0.05 or 0.25 times the vertical reaction otherwise.
- Seismic Performance Zone 2 – use appropriate *R*-factors (*R*/2 may be used for foundation but $R/2 \geq 1.0$) for design or consider possible column overstrength.

- Seismic Performance Zone 3 – use appropriate *R*-factors for column design and R=1.0 or column plastic hinging for foundation design.
- Additional horizontal design inertial forces equivalent to half of A_S shall be applied to the footing or pile cap when column plastic hinging is used for foundation design.

(3) *Foundation Requirements:* The requirements for the design of spread foundation and pile foundation are given in this section. Yielding of the foundation shall not be allowed under earthquake where the foundation yielding is defined as any of: (a) yielding of foundation members, (b) yielding of the ground, and (c) uplift of the foundation. Moreover, the effects of seismically unstable ground shall be considered in the design of foundation.

(4) *Longitudinal Restrainers and Hold-Down Devices:* Longitudinal restrainers shall be provided in accordance with the unseating prevention system and hold down devices shall be provided when the uplift due to longitudinal seismic forces exceeds 50% of the reaction due to permanent loads.

(5) *Bearing Support System:* The bearing support system is classified into:

- Type A Bearing Support – This bearing support system (typically applied in the Philippine bridges) shall be designed to resist the horizontal and vertical forces under Level 1 earthquake ground motion (EGM) and shall jointly resist the horizontal and vertical forces due to Level 2 EGM with the device for limiting excessive displacement.
- Type B Bearing Support – This bearing support type shall be designed to resist the horizontal and vertical forces due to Level 1 and Level 2 EGM. The bearing support system such as seismic isolation bearing and the elastic bearing that distributes the horizontal forces to the substructures shall conform to this provision.

10.2.6 Section 6 : Effects of Seismically Unstable Ground

The effects of the unstable ground shall be taken into account in the verification of seismic performance of a bridge when the ground is expected to be in an unstable state during an earthquake. Unstable ground is defined as an extremely soft soil layer in seismic design, or a sandy layer affecting the bridge due to the liquefaction and lateral spreading.

(1) *Geotechnical Parameters:* The geotechnical parameters for extremely soft soil layer which may affect the bridge during liquefaction shall be reduced according to this section.

(2) *Liquefaction Assessment:* The determination of necessity for assessment of liquefaction is shown in Figure 10.2.6-1.

(3) *Liquefaction-Induced Lateral Spreading:* The ground with both of the following conditions shall be treated as a ground with possible lateral movement affecting the bridge:

1. Ground within a distance of less than 100 m from a water front in a shore area formed by a revetment with an elevation difference of 5 m or more between the water bottom and the ground surface behind.
2. Ground with a sandy layer thicker than 5 m that is assessed as a liquefiable layer according to the provisions in Article 6.2.3 and is distributed somewhat widely in the area of the water front.

The model for calculating lateral movement forces is illustrated in Figure 10.2.6-2.

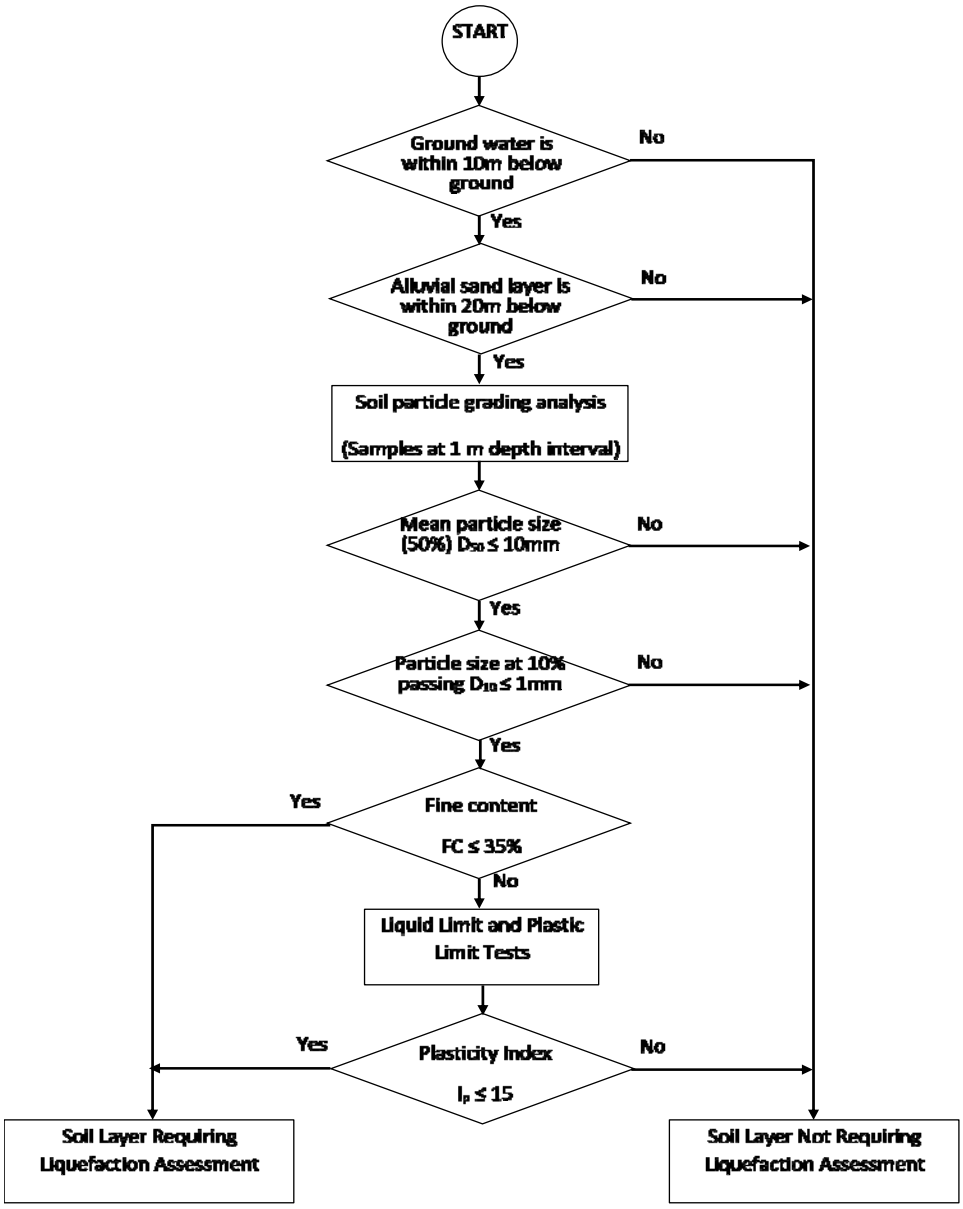


Figure 10.2.6-1 Determination of Liquefaction Assessment Necessity

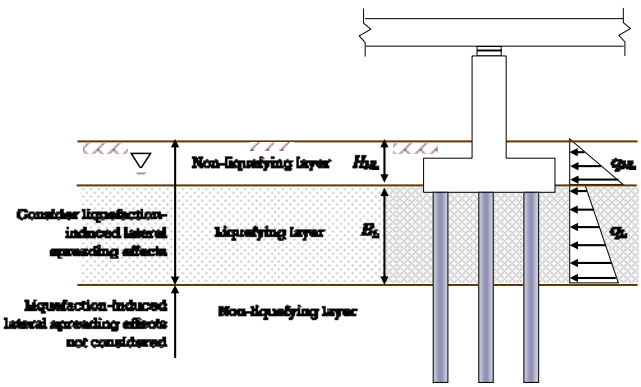


Figure 10.2.6-2 Model to Calculate Lateral Movement Forces

10.2.7 Section 7 : Requirements for Unseating Prevention System

An unseating prevention system consists of the seating length of the girder at the support, unseating prevention device, device limiting excessive displacement, and device to prevent the superstructure from settling (limiting vertical gap in superstructure). These components shall be appropriately selected in accordance with the bridge type, type of bearing supports, ground conditions, and other factors deemed necessary by DPWH. Figure 10.2.7-1 illustrates the mechanism of unseating prevention system while Figure 10.2.7-2 shows the fundamental principles of unseating prevention system.

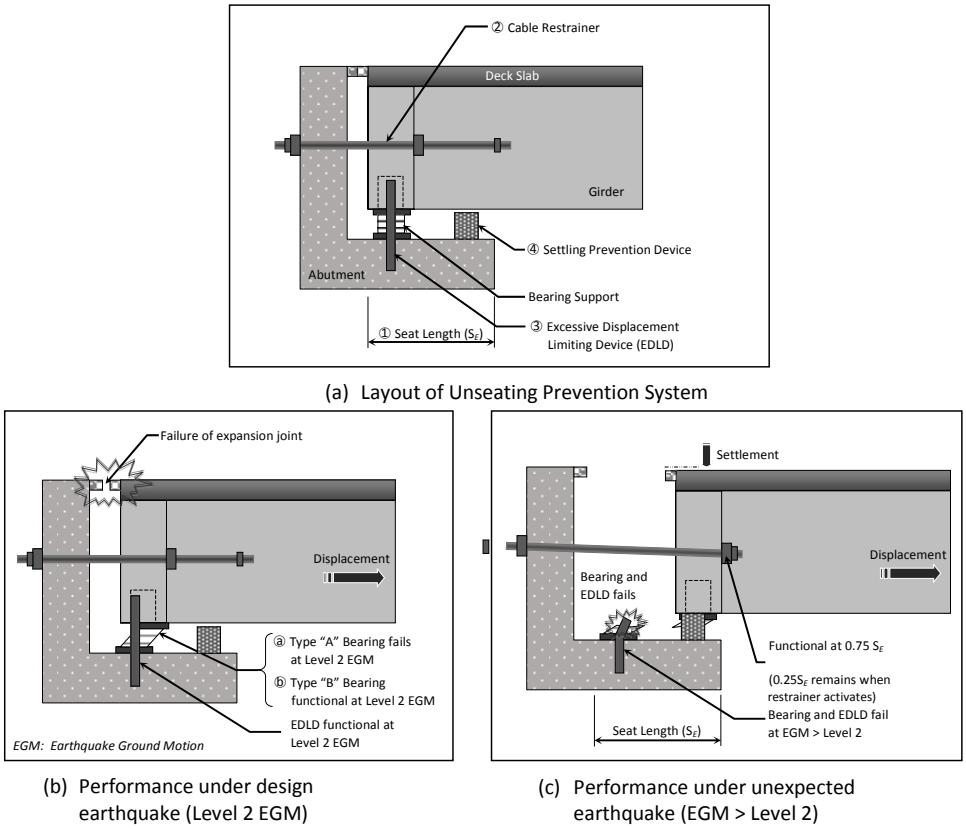


Figure 10.2.7-1 Mechanism of Unseating Prevention System

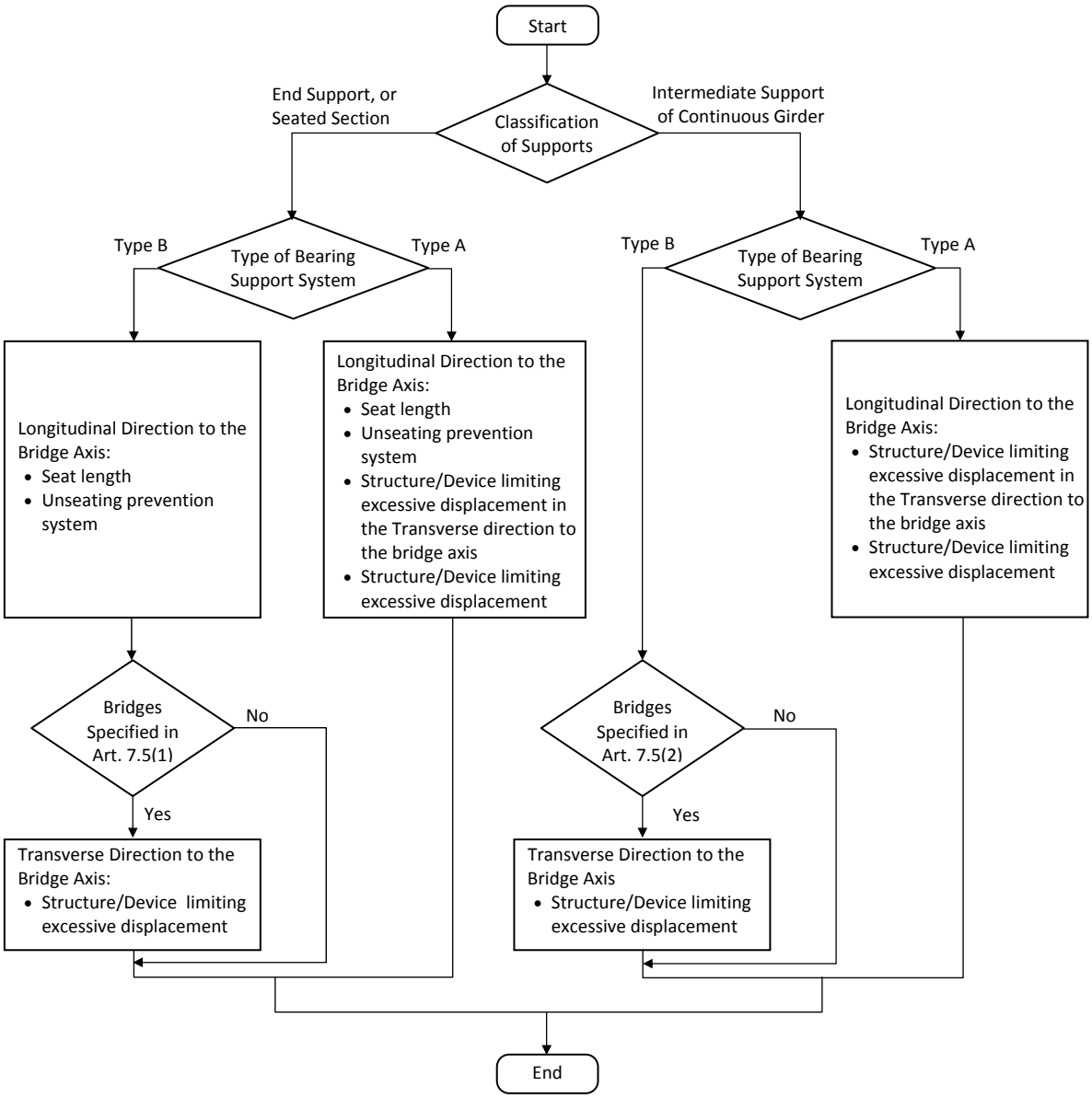


Figure 10.2.7-2 Fundamental Principles of Unseating Prevention System

10.2.8 Section 8 : Requirements for Seismically Isolated Bridges

The introduction of seismic isolation design shall be considered to increase the natural period of vibration, such that the bridge is subjected to lower earthquake forces, as well as to increase the energy absorption capacity of the bridge for both normal and earthquake conditions. However, seismic isolation shall not normally be adopted for a bridge meeting the following conditions:

1) The bridge is located in a soil layer for which the seismic geotechnical parameter determined from Section 6 of these Specifications is zero.

2) The bridge has fairly flexible substructures and long fundamental natural period.

3) The bridge is located in a soft soil layer with long natural period that may cause resonance with the bridge if seismic isolation is introduced.

4) The bridge has uplift at bearing supports.

The performance requirements for isolation bearings include:

(1) The variation of the equivalent stiffness of an isolation bearing shall be within ± 10% of the value obtained from an equivalent linear model. On the other hand, the equivalent damping ratio of an isolation bearing shall be greater than the value calculated from an equivalent linear model.

(2) An isolation bearing shall be stable when subjected to repeated load corresponding to the design displacement u_B, which is calculated in the verification for Level 2 Earthquake Ground Motion.

(3) An isolation bearing shall possess positive tangential stiffness within the range of the design displacement u_B, calculated in the verification for Level 2 Earthquake Ground Motion.

(4) An isolation bearing shall generally prevent the occurrence of residual displacement affecting post-earthquake bridge functions.

(5) The equivalent stiffness and the equivalent damping ratio of an isolation bearing shall be stable for the environmental conditions including repeated live loads and temperature changes.

10.3 Outline of the Seismic Design Calculation Example using the Bridge Seismic Design Specifications (BSDS)

In order to guide the design engineers in utilizing the Bridge Seismic Design Specifications (BSDS), a seismic design calculation example is developed as an accompanying volume of the BSDS. The design example covers the basic principles and processes of seismic design in accordance with the BSDS.

10.3.1 Policy in the Development of Seismic Design Example

The basic policies in the development of the design examples are as follows:

(a) The primary user is assumed to be the DPWH bridge design engineers and consultant's bridge design engineer level undertaking bridge design in the Philippines. As such, the basic principles of design shall not be covered.

(b) An understanding the basic principles, concepts and procedure of seismic design for new bridges is be encouraged.

(c) The seismic design example will refer to the design provisions of the BSDS and shall also refer to AASHTO Specifications for design requirements not covered in the BSDS.

(d) The "Seismic Design Calculation Examples" is prepared in general, taking into account the technical experience and level of Philippine engineers.

10.3.2 Outline of Seismic Design Example

The seismic design example basically follows the procedure of the BSDS and attempts to explain the provisions related to the design of substructures and foundations. Although much effort has been given to the application of the design example to BSDS, the design example is limited to some extent in the basic design of single piers. The outline of the seismic design example is illustrated in Figure 10.3.2-1 below.

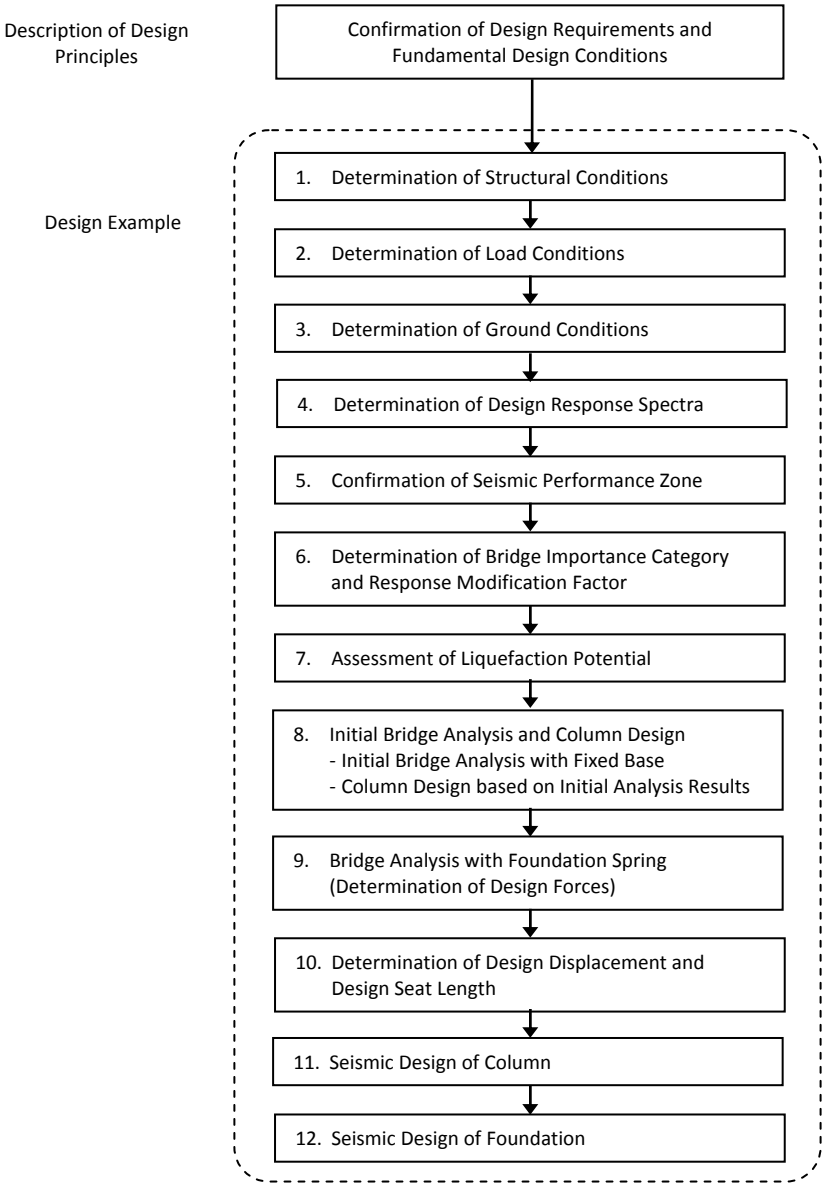


Figure 10.3.2-1 Outline of Seismic Design Example

(1) Fundamental Design Conditions

The fundamental design conditions for seismic design is given in Sections 1 to 5 of the BSDS and in the accompanying seismic design calculation examples, the following design conditions are summarized:

1. Bridge Importance (Operational Class)	• The bridge example is assumed to be part of the major arterial network belonging to the Pan-Philippine Highway Road and as such is classified as Critical (OC-I)
2. Response Modification Factor (R-factor)	• For a single column (example pier), the R-factor under Essential Category is 2.0
3. Seismic Performance Requirement (SPL)	• Under Level 2 Ground Motion Earthquake for Critical bridges, the seismic performance requirement shall be SPL-2
4. Load Combination	• The Extreme Event I load combination with 50% live load
5. Unit Weight	• Typical unit weights as given in AASHTO are applied
6. Material Properties	• AASHTO material properties are used
7. Ground Condition for Seismic Design	• Based on the soil profile given, the characteristic value of the ground is 0.232 sec which corresponds to Type II Ground Type
8. Design Response Spectrum	• The 3-point response spectrum based on seismic hazard maps of PGA, 0.2-sec response and 1.0-sec response is used to develop the design response spectrum. The site factors corresponding to Type II soil is used to consider the site amplification under earthquake
9. Seismic Performance Zone	• Considering the potential for liquefaction and the S_{D1} value of 0.64, the design should comply with seismic performance zone SPZ-4
10. Analysis Requirements	• For bridges under SPZ-4 and Critical category, a multi-mode elastic method of analysis is required.
11. Analysis Model	• In this seismic design example, a single-degree-of-freedom model is used to simplify the analysis. • Since plastic hinging of the column is expected, the cracked section property of column is used in the analysis model. • Initial model to determine the foundation requirements shall be to assume the piers (columns) to have a fixed base. • The dynamic foundation springs shall be used in the subsequent analysis
12. Design Forces	• The design forces correspond to SPZ-4. R-factors are used for the orthogonal forces combination. • For foundation design, the plastic hinging forces generated by the column is applied with additional inertial forces from the pile cap.
13. Column Section Design	• The factored resistance of the column is made greater than the demand forces. • Detailing for shear resistance or concrete confinement is based on the AASHTO requirements.
14. Assessment of Liquefaction Potential	• The potential for liquefaction is assessed based on the ratio of the dynamic shear strength resistance and the seismic shear stress induced by earthquake. • The ratio R/L is determined as 0.467 which indicates that the ground has potential for liquefaction.
15. Design of Pile Foundation	• Static soil springs and pile capacities as specified in the BSDS are used for foundation design model. • Demand design forces for the pile body are determined from the BSDS recommendations.

(2) Bridge Layout for Seismic Design Example

The bridge design example for seismic analysis is illustrated in Figure 10.3.2-2 below. The bridge structure is a series of single span bridges with 30m-long spans between supports with 2-lane travelways and a total of 10.5m bridge width. The superstructure consists of 3 lines of steel I-girder with concrete deck. Moreover, the substructure consists of single circular columns 1.9m in diameter and 10m high from the column base to the top of the coping. The foundation is a pile-type foundation consisting of 5-1.0m diameter piles and 12m long. The superstructure supports are fixed on one end and movable on the other end.

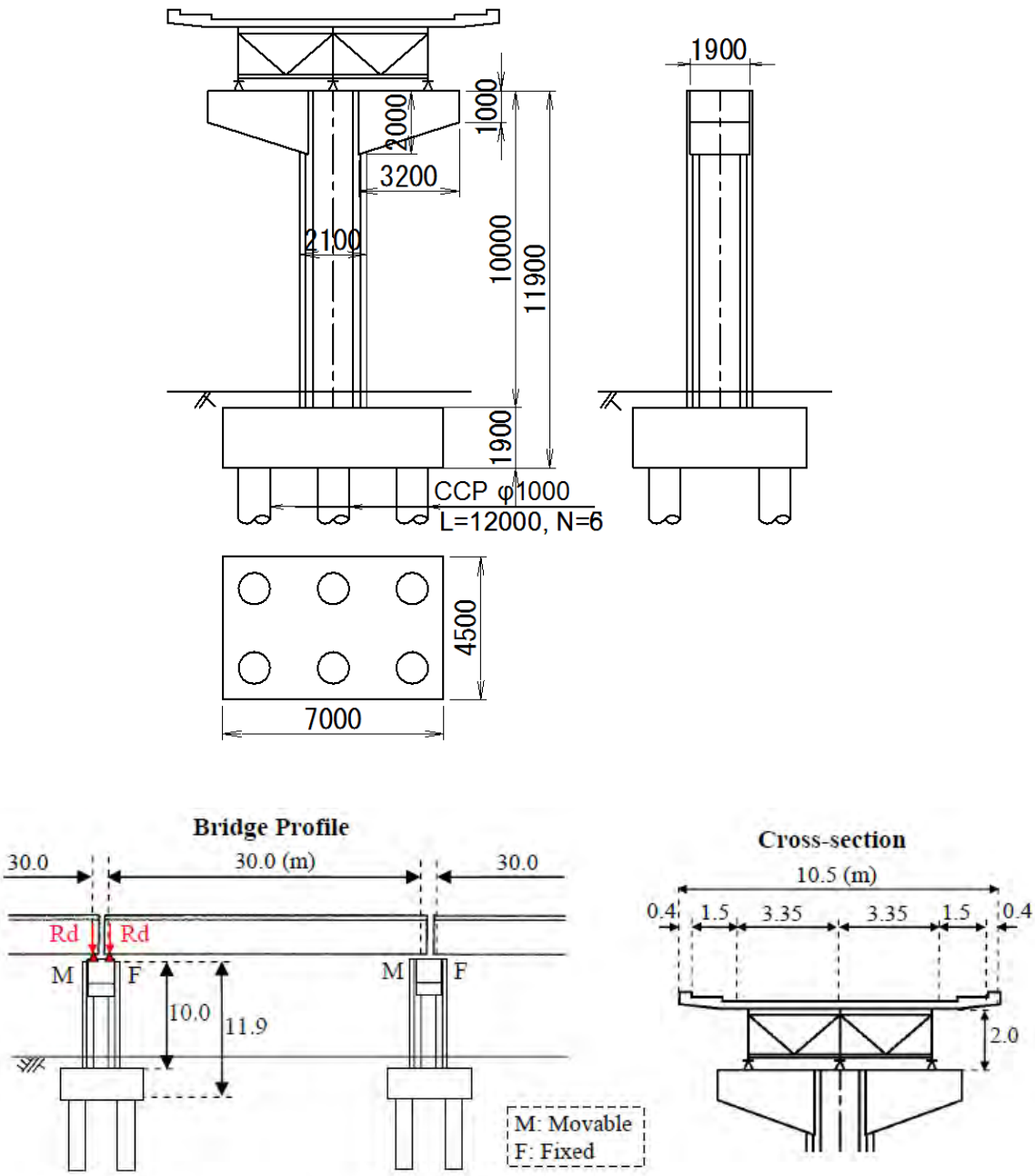


Figure 10.3.2-2 Bridge Design Example Layout

(3) Ground Condition for Design Example

The ground condition for the proposed bridge example is shown in Figure 10.3.2-3. As seen in the figure, the tuff bearing layer is about 11m from the ground surface overlain by clayey sand and silty fine sand.

The characteristic ground value is calculated as T_G = 0.232 which classifies the ground profile to be of Type II.

Layer symbol	Layer Thickness (m)	Soil Parameters: N-value Blows	N-value Ave.	γt (kN/m²)	FC (%)	D50 (mm)	C (kN/m²)	φ (°)	Eo (kN/m²)	Vsn (m/sec)
Bs	1	12	12	17	0.9	0.74	0	35	8,400	183
As	5	7	11	17	17.3	0.14	0	34	7,700	178
		6		17	28.0	0.12				
		8		17	12.0	0.21				
		15		17	7.3	0.42				
		21		17	7.3	0.20				
Ac	4	7	7	15	58.1	-	44	0	4,900	191
		9		15	77.1	-				
		6		15	66.9	-				
		8		15	51.0	-				
WGF	1	28	28	17	0.2	2.38	-	37	19,600	292
GF	10	50	50	21	0.5	0.60	173	21	39,530	292
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
		50		21	-	-				
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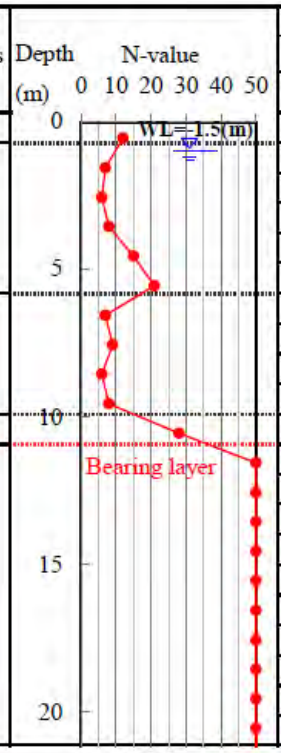


Layer Symbol	Soil characteristics
Bs	Medium sand; brown colored; and with broken shell fragments
As	Silty fine sand; soft or loose; relatively high water content; and mostly gray colored
Ac	Sandy clay or clayey sand; dark-gray colored; and moderate water content
WGF	Weathered rock; strongly weathered (probably tuff); gray colored; and sand-like
GF	Tuff breccia, tuffs, and tuffaceous sandstones; brownish-gray colored; 11 m 17 m: strongly weathered portion; and below 17 m: fresh and/or welded portion
	Fine sand; with broken shell fragments; including fines and gravel; and dark-gray or brownish-gray colored
	No core recovered; and probably fine sand
	Strongly welded tuff; and black colored

Figure 10.3.2-3 Ground Condition for Foundation Design

In Figure 10.3.2-4, the soil layer "As" satisfies the condition for potential liquefaction. In this case, it is analyzed for potential liquefaction and following the procedure outlined in the BSDS, it is determined that the FL = R/L = 0.467 which indicates that this soil layer is liquefiable. Moreover, based on the average R-value which is 0.317, the soil parameter reduction factor "DE" is determined to be 2/3.

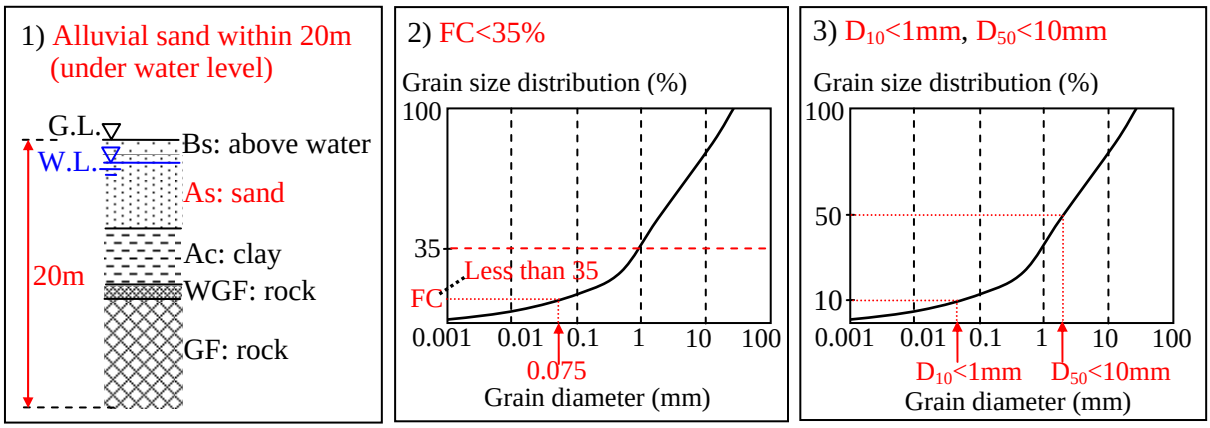


Figure 10.3.2-4 Characteristics of Soil Layer "As"

(4) Design Acceleration Response Spectra

The design acceleration response spectra is determined based on the acceleration coefficients derived from the hazard maps shown in Figure 10.3.2-5.

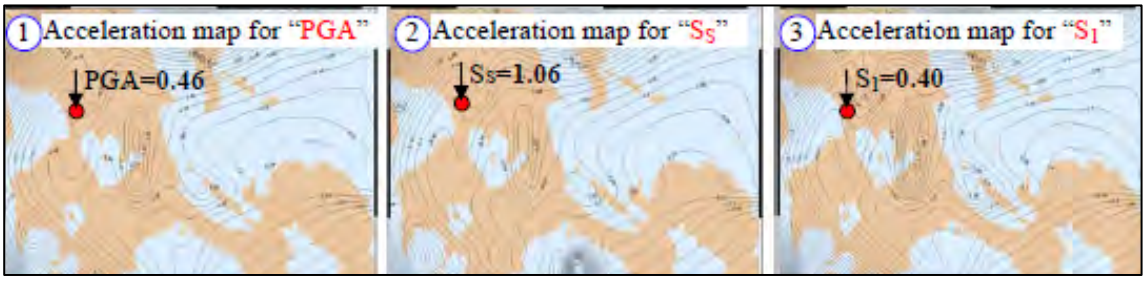


Figure 10.3.2-5 Acceleration Coefficients for Site

In order to plot the design response spectrum, the coefficients above are multiplied by the corresponding factors based on Type II ground profile. The design response spectrum obtained as shown in Figure 10.3.2-6

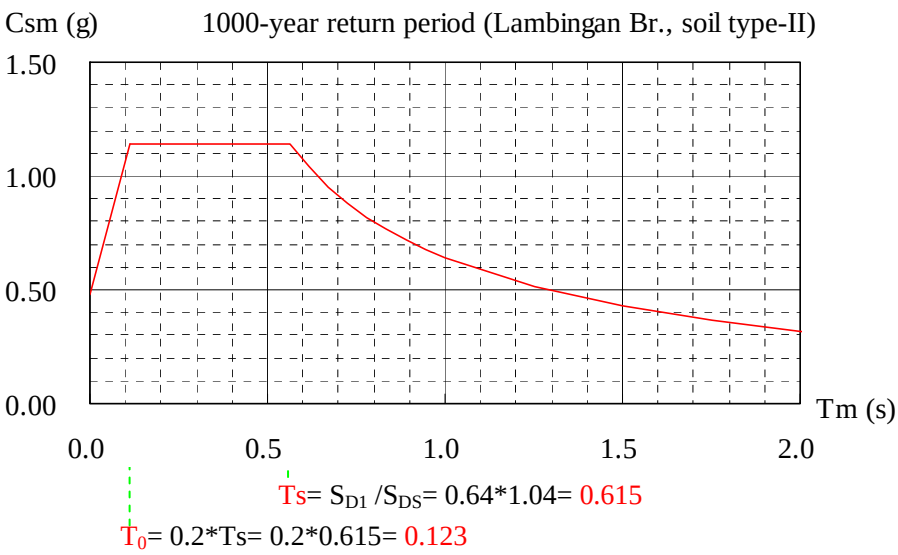


Figure 10.3.2-6 Design Acceleration Response Spectrum

(5) Seismic Performance Zone

Since S_{D1} is determined to be 0.64, the site is categorized under seismic performance zone SPZ-4 ($S_{D1} \geq 0.50$).

(6) Response Modification Factor

For bridges under critical operational category, the R-factor is taken as 2.0

(7) Dynamic Spring Constants

The dynamic spring properties for the pile foundation is calculated based on the procedure recommended in the BSDS using the dynamic modulus of ground deformation, the dynamic shear modulus and the dynamic Poisson's ratio. This is determined by iteration of the values of 1/β and K_H. In this example, the calculated values for the pile foundation spring properties are given in Figure 10.3.2-7.

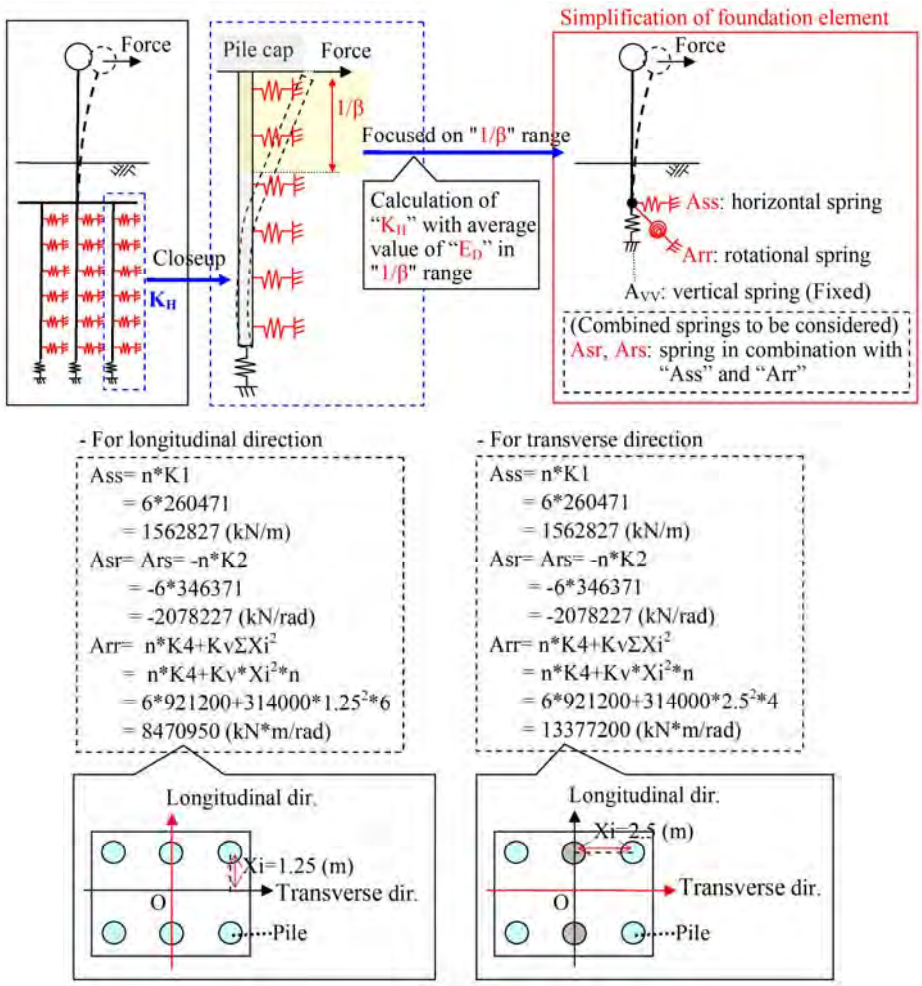


Figure 10.3.2-7 Pile Foundation Model and Spring Properties

(8) Bridge Analysis Model

The seismic design example is simplified by assuming a single-degree-of-freedom model for the one-span vibration unit. The simplification is illustrated in Figure 10.3.2-8.

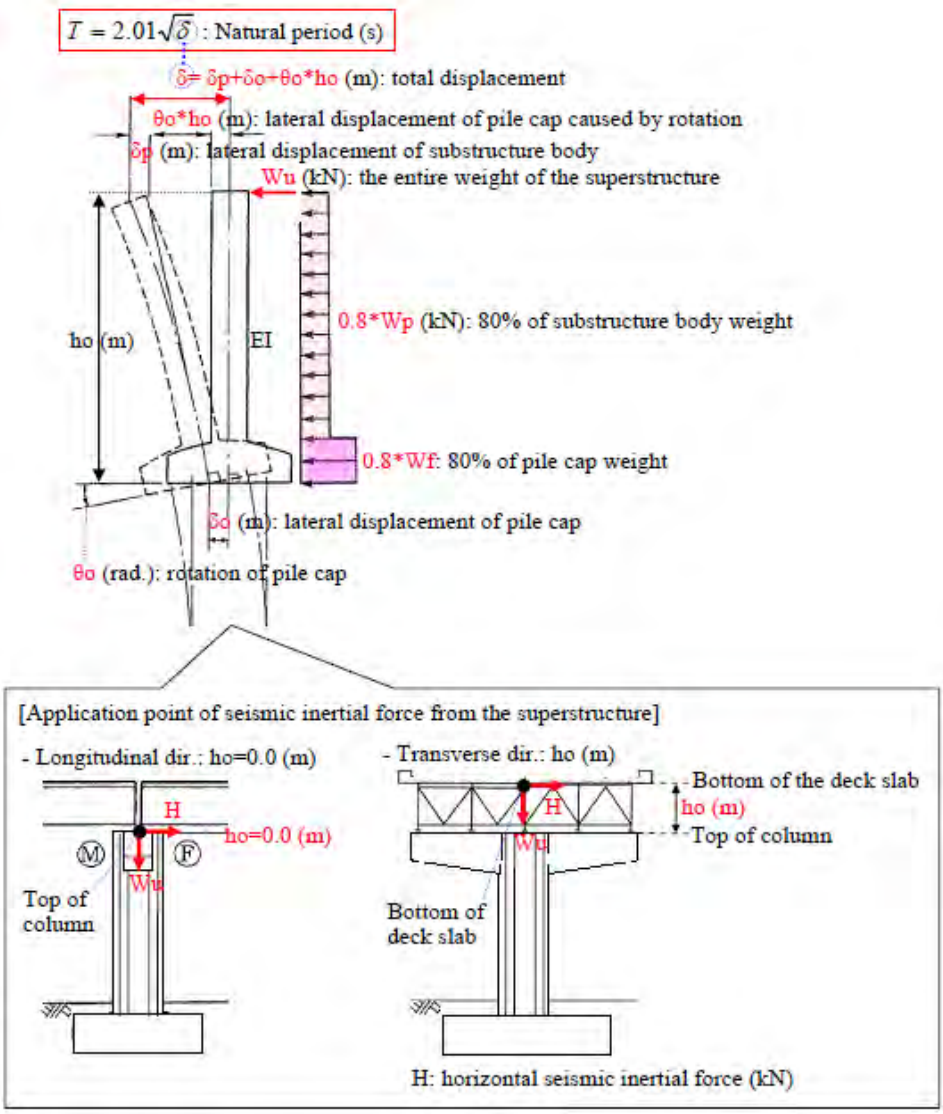


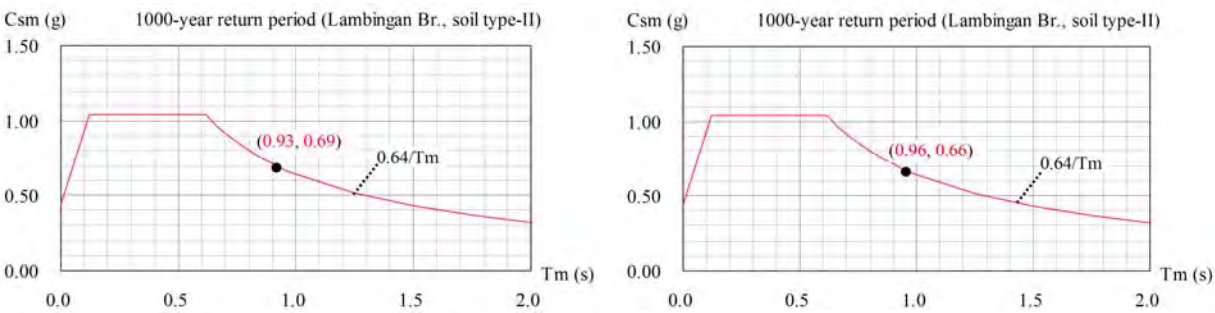
Figure 10.3.2-8 Pier Modeled as a Single-Degree-of-Freedom Vibration Unit

Using the simplified model, the natural period is calculated and determined to be:

Longitudinal Direction : 0.93 sec

Transverse Direction : 0.96 sec

The design seismic coefficients are determined by plotting the calculated natural periods in the design acceleration response spectrum curve determined earlier in Section 10.3.2 (4). Thus, by using the natural periods determined for the longitudinal and transverse directions, the design seismic coefficients are given in Figure 10.3.2-9.



(a) Longitudinal Direction (b) Transverse Direction

Figure 10.3.2-9 Design Seismic Coefficients

(9) Column Design (Flexure and Shear)

The design forces for the column as specified in the BSDS shall be a combination of the earthquake demand forces generated from the two orthogonal directions of the bridge. The combination of forces in the longitudinal and transverse direction is illustrated in Figure 10.3.2-10. Using 2-rows of ϕ36mm (spacing at 135mm for the 1st row and 270mm for the 2nd row) as the column main reinforcement, the factored resistance is determined to be Mr = 19456 kN-m with the column interaction diagram shown in the figure below. The demand forces are calculated to be less than the column factored capacity.

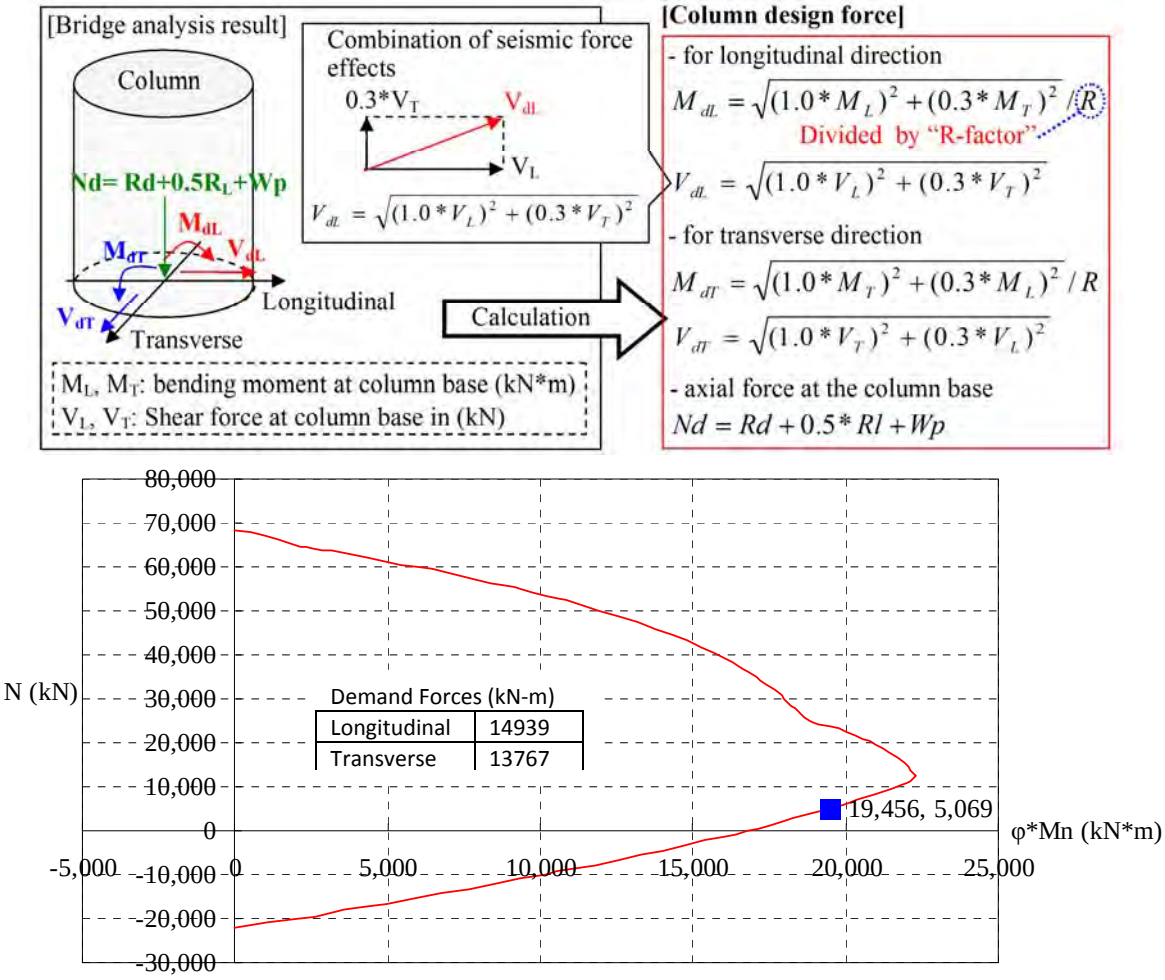


Figure 10.3.2-10 Combination of Column Design Forces

The column shear capacity is likewise checked against the shear demand forces and the minimum reinforcement required for confinement, as summarized in Table 10.3.2- 1 below.

Table 10.3.2-1 Column Shear Design

Reinforcement	Demand Shear (kN)		Column Shear Capacity (kN)	Minimum ρ for Confinement	ρ Provided
2-bundle ϕ20 @100mm o.c.	Longitudinal	3011	5710	0.0110	0.0116
	Transverse	2903			

(10) Pile Foundation Design

In a similar manner described earlier, the spring constants for design of the pile foundation are calculated based on the BSDS recommendations and the values determined as shown in Figure 10.3.2-11.

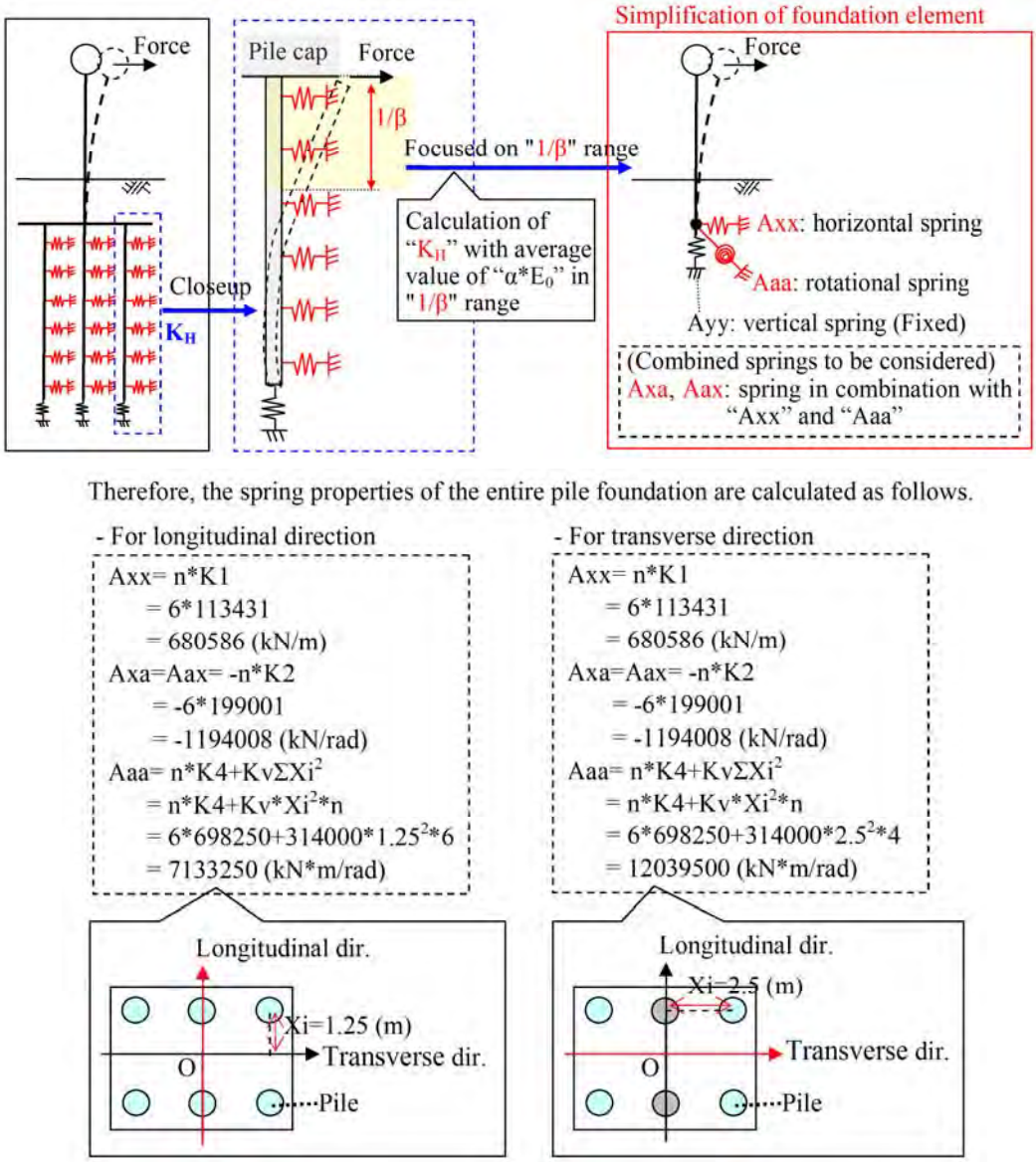
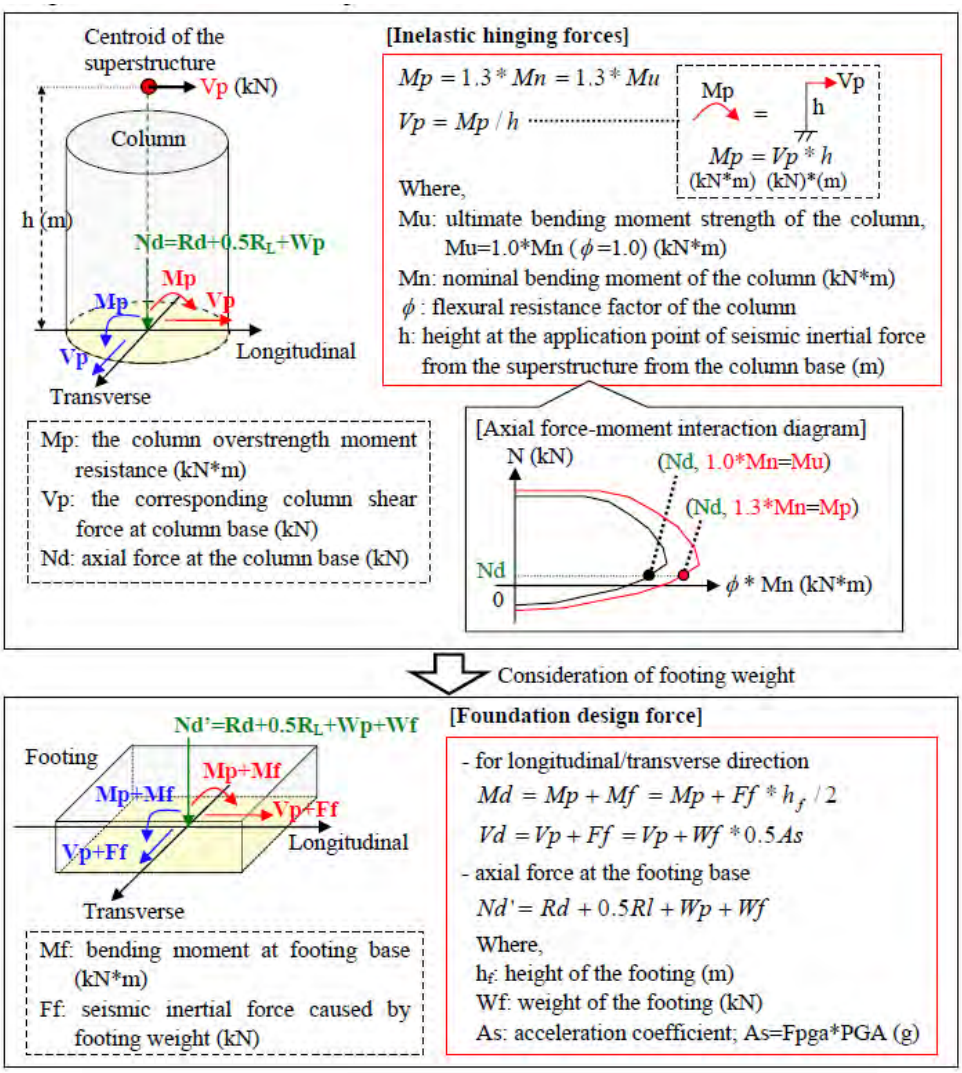


Figure 10.3.2-11 Pile Foundation Model and Pile Spring Constants

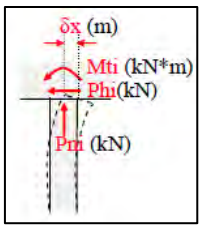
The design forces for the foundation are taken from the overstrength capacity of the column by invoking plastic hinges being formed at the base of the column. The determination of the foundation design forces is summarized and illustrated in Figure 10.3.2-12.



Forces	Column Overstrength	Foundation Design Forces
Axial (kN)	3800	6,991
Plastic Moment, Mp (kN-m)	25,293	25,593
Plastic Shear, Vp (kN)	Long-2,529; Trans-2,108	2,424

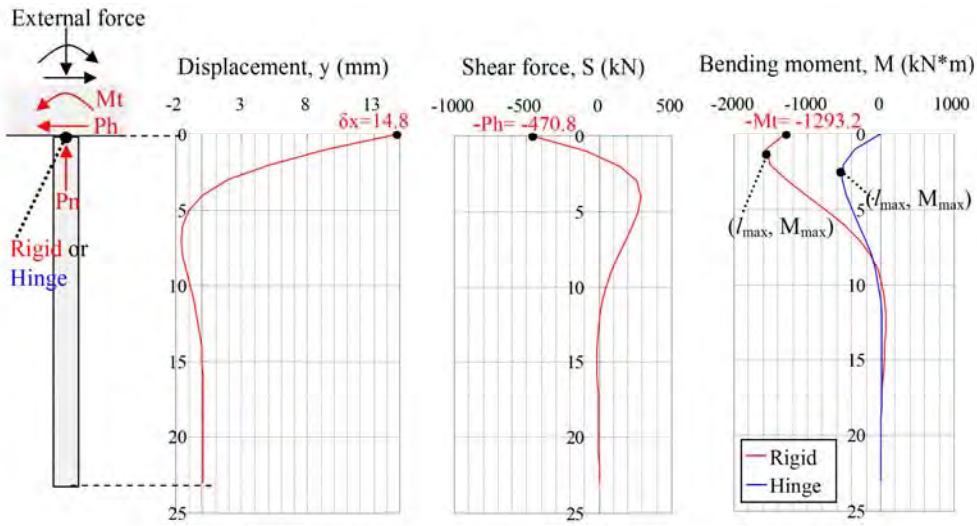
Figure 10.3.2-12 Foundation Design Forces

From the demand design forces at foundation level, the reaction forces at the pile heads in the longitudinal and transverse directions are determined to be:

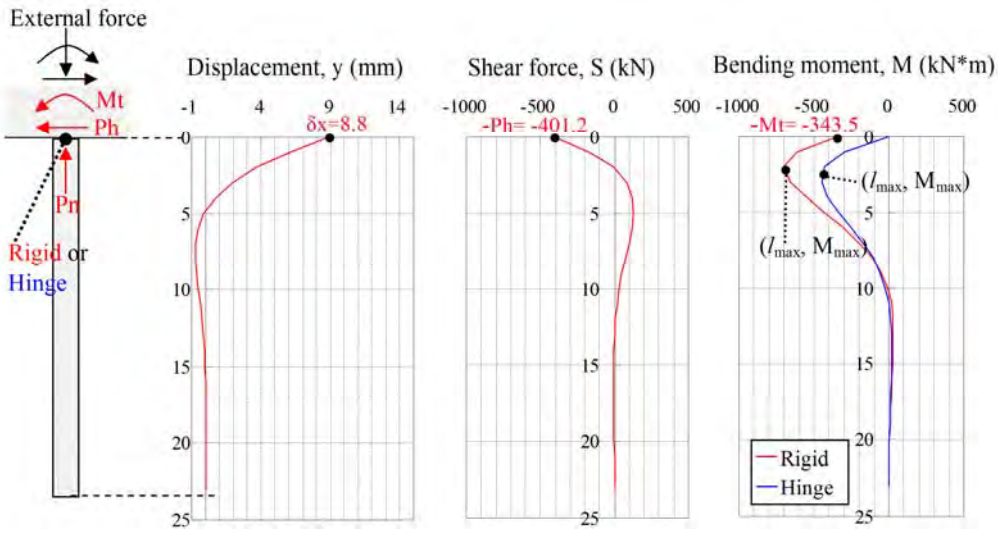


Longitudinal Direction				
Row	Xi (m)	Pni (kN)	Phi (kN)	Mti (kN-m)
1	-1,250	-1,299.2	470.8	1293.2
2	1,250	3,465.8	470.8	1293.2
Transverse Direction				
Row	Xi (m)	Pni (kN)	Phi (kN)	Mti (kN-m)
1	-2,500	-1,271.7	401.2	343.5
3	2,500	3,438.3	401.2	343.5

Pile Geotechnical Capacity	
Bearing (Compression), kN	4,009.6
Pull-out (Tension), kN	-1,678.2



(a) Longitudinal Direction



(b) Transverse Direction

Figure 10.3.2-13 Reaction Force and Displacement at Pile Body

Based on the section demand forces, the pile reinforcement is determined to be 18-ϕ36 for the main reinforcing bars and ϕ16 spiral @100mm o.c. The pile interaction diagram is shown in Figure 10.3.2-14.

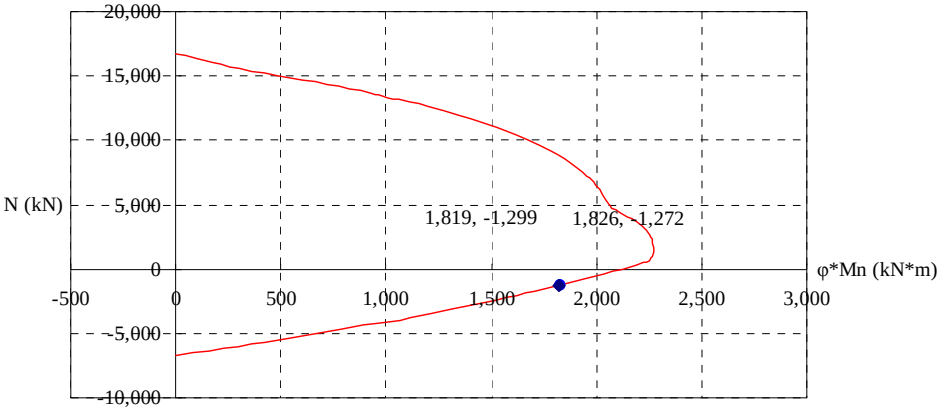


Figure 10.3.2-14 Pile Section Interaction Diagram

10.4 Comparison between the DPWH Existing Design with the Bridge Seismic Design Specifications (BSDS) Using the Proposed Design Acceleration Response Spectra

New specifications are proposed in view of the deficiencies in the current seismic design practice of bridges in the Philippines. The proposed Bridge Seismic Design Specifications (BSDS) which is based in the latest AASHTO LRFD design specifications, however, have several design requirements which differs from the previous design practice of DPWH, namely:

- The use of response acceleration spectra based on the PGA, short-period and long-period acceleration response from the developed seismic hazard maps, as opposed to the current practice of using the AASTO spectra based on four soil type classification.
- The use of the proposed seismic hazard map for the entire Philippines based on the probabilistic seismic hazard analysis of past records of earthquake as opposed to the current use of 0.4g and 0.2g PGA to be applied in the design response spectra.
- The increase in return period of the design earthquake from 500-years (current) to 1,000-years (BSDS).
- The reduction of R-factors to almost half for Critical and Essential bridges as opposed the current R-factors.
- The application of LRFD (load and resistance factors) as opposed to the current LFD (load factors).

10.4.1 Comparison Objective

The objective of the comparison study is to examine the:

- difference in design output of using the current DPWH design practice for bridges and the proposed BSDS, and
- effect of increasing the design earthquake return period from 500-years to 1,000-years.

10.4.2 Comparison Condition

The following design conditions are used in the comparison:

- Bridge Type : simply supported composite steel I-girder with concrete deck
- Span Length : 30 m
- Total Road Width : 10.5 m
- Skew Angle : 90 deg
- Pier Type : single circular column
- Pier Height : 11.9 m (column height: 10.0 m)
- Foundation Type : ϕ1000mm cast-in-place concrete pile foundation
- Superstructure : Center of mass at 2.0 m above the top of column
- Dead Load : Reaction force, Rd = 2,900 kN/m
- Live Load : Reaction force, Rl = 1,800 kN/m

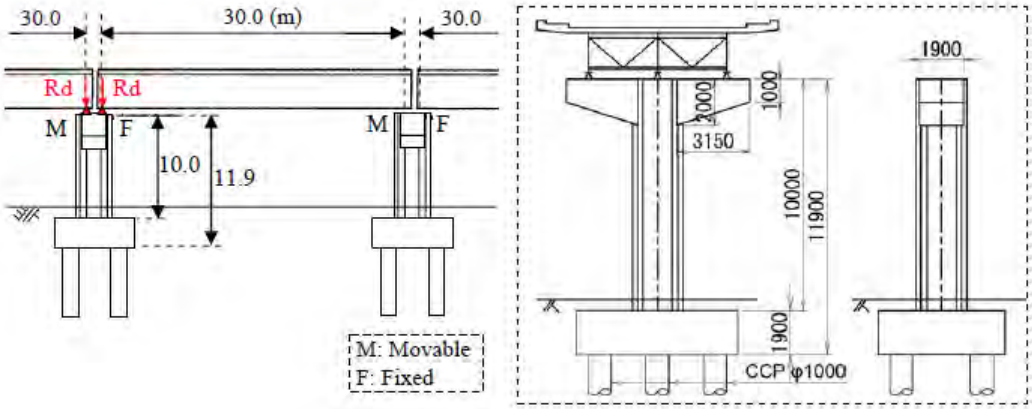


Figure 10.4.2-1 Pier Layout for Comparison Study

The ground condition is Ground Type II with the same soil layer profile as the seismic design example. The design acceleration response spectra for comparison, as shown in Figure 10.4.2-2 includes (a) DPWH/NSCP spectrum for AASHTO Type II soil, (b) BSDS 500-year return spectrum for Type II soil, and (c) BSDS 1,000-year return spectrum for Type II soil.

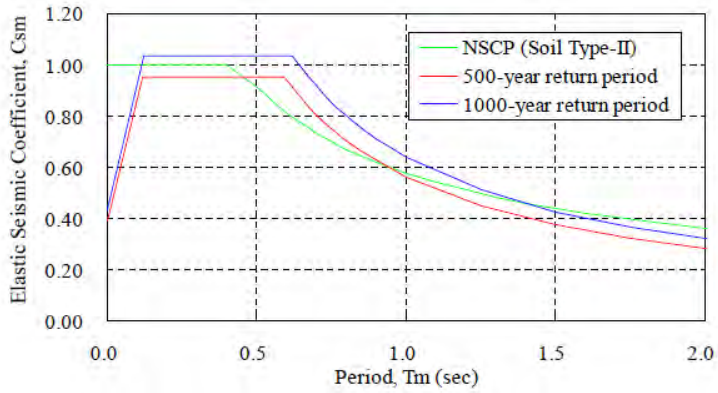


Figure 10.4.2-2 Design Acceleration Response Spectra (3-Cases)

10.4.3 Cases for Comparison

The following five cases were used for the comparative study.

Table 10.4.3-1 Cases for Comparison

Case	Applied Code	Design Earthquake Return Period	Column R-factor	Column Diameter (m)	No. of Piles	Resistance Factor		Column Section Property	Displace-ment Check[3)]
						Flexure	Shear		
1	DPWH/ NSCP[1)]	500-yr (NSCP)	3.0 (Essential/ Critical)	2.0	5	0.75	0.85	Uncracked	Not required
2	BSDS[2)]	500-yr (BSDS)	2.0 (Essential)	2.1	6	0.75	0.85	Cracked	Required
3	BSDS	1,000-yr (BSDS)	2.0 (Essential)	2.1	6	0.90	0.90	Cracked	Required
4	BSDS	500-yr (BSDS)	1.5 (Critical)	2.4	6	0.75	0.85	Cracked	Required
5	BSDS	1,000-yr (BSDS)	1.5 (Critical)	2.4	8	0.90	0.90	Cracked	Required

Notes: [1)] The DPWH/NSCP is the design code currently applied for seismic design of bridges using the Philippine PGA map (2-zone), the AASHTO design response spectra, the R-factors for 2-Importance Class and assuming a design earthquake with 500-year return.

[2)] The BSDS is the proposed bridge seismic design specifications with the generated seismic hazard maps and the 3-Operational Classification for bridges.

[3)] Verification formula: Δ*Pu < 0.25*φ*Mn

10.4.4 Results of Comparison

The results of comparison is presented in Table 10.4.4-1 and summarized as follows:

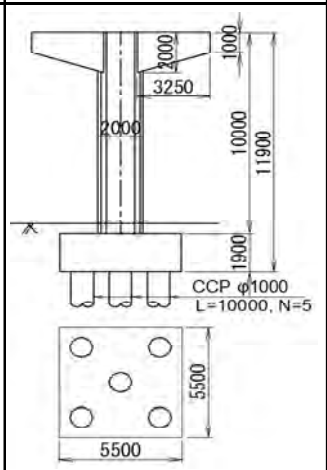
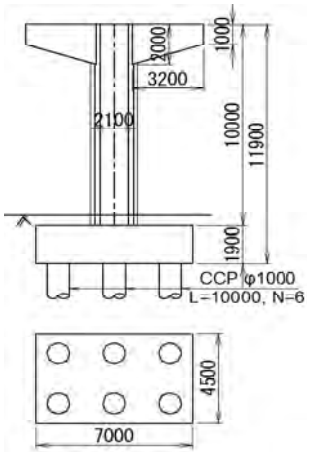
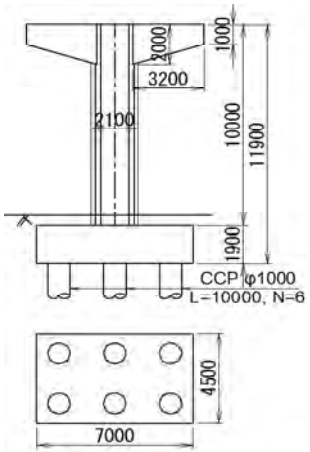
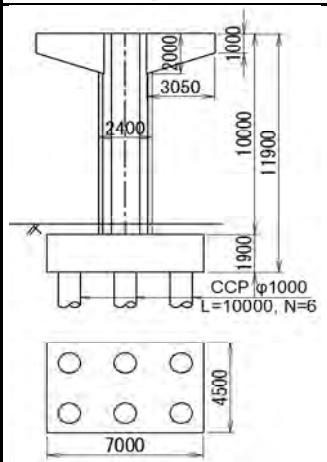
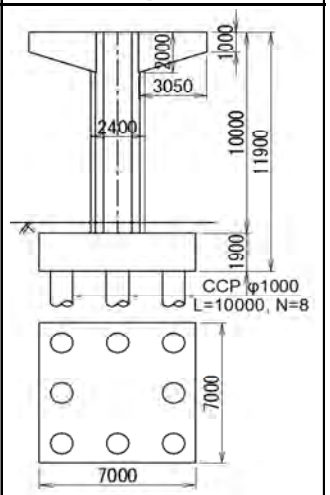
- The current DPWH/NSCP bridge design specifications are used as the base of comparison with a cost factor ratio of 1.0 (Case 1).
- When considering the “Essential” bridge operational classification (R-factor reduced from 3.0 to 2.0), the cost factor increases to 1.18 for a 500-year design earthquake and 1.21 for a 1,000-year design earthquake.
- When considering the “Critical” bridge operational classification (R-factor reduced from 3.0 to 1.5), the cost factor increases to 1.25 for a 500-year design earthquake and 1.65 for a 1,000-year design earthquake.

- When the return period for the design earthquake is increased from 500-year to 1,000-year, the increase in relative cost for ‘Essential” bridges is only 2% while for “Critical” bridges is about 32%

From the results of the comparative study, it can be deduced that:

- It is more practical to apply the proposed seismic hazard maps and the design response spectra of the proposed BSDS since the earthquake design forces are based on the possible sources of ground motion rather than assuming a single PGA as recommended in the existing specifications.
- The increase in relative cost using operational class “Essential” for DPWH bridges varies by about 20% from the existing importance class (acceptable). However, the increase in relative cost using the “Critical” operational class is 25-65%; it can be used for very important bridges as defined in the BSDS.

Table 10.4.4-1 Results of Comparative Study

Study Case		Case-1		Case-2		Case-3		Case-4		Case-5	
Applied Spectra		NSCP		BSDS		BSDS		BSDS		BSDS	
Return period		500-year		500-year		1000-year		500-year		1000-year	
R-factor		3.0 (Essential/Critical)		2.0 (Essential)		2.0 (Essential)		1.5 (Critical)		1.5 (Critical)	
Resistance factor	Flexural	φ= 0.75		φ= 0.75		φ= 0.9		φ= 0.75		φ= 0.9	
	Shear	φ= 0.85		φ= 0.85		φ= 0.9		φ= 0.85		φ= 0.9	
Column stiffness		Uncracked section		Cracked section		Cracked section		Cracked section		Cracked section	
Displacement check		Not required		Required[1)]		Required[1)]		Required[1)]		Required[1)]	
Dimension											
Reinforcement (Column)		154 kg/m3		186 kg/m3		203 kg/m3		181 kg/m3		208 kg/m3	
Seismic coefficient	Long. dir.	Csm= 0.60 (T= 0.94 sec)		Csm= 0.60 (T= 0.93 sec)		Csm= 0.69 (T= 0.93 sec)		Csm= 0.72 (T= 0.78 sec)		Csm= 1.02 (T= 0.63 sec)	
	Trans. dir.	Csm= 0.57 (T= 1.03 sec)		Csm= 0.47 (T= 0.96 sec)		Csm= 0.66 (T= 0.96 sec)		Csm= 0.55 (T= 0.83 sec)		Csm= 0.72 (T= 0.78 sec)	
Column Capacity	Flexure	Md= 8499 < 10451 (= 0.75*Mn)		Md= 11825 < 14797 (= 0.75*Mn)		Md= 14939 < 194569 (= 0.9*Mn)		Md= 19212 < 22071 (= 0.75*Mn)		Md= 27395 < 30723 (= 0.9*Mn)	
	Shear	Vd= 2572 < 5121 (= 0.85*Vn)		Vd= 2599 < 5393 (= 0.85*Vn)		Vd= 3011 < 5710 (= 0.9*Vn)		Vd= 3252 < 6209 (= 0.85*Vn)		Vd= 4626 < 6574 (= 0.9*Vn)	
	Disp.	-		δ= 0.91 < 0.97		δ= 1.24 < 1.28		δ= 0.62 < 1.45		δ= 0.89 < 2.02	
Stability	Bearing	$P_{n\text{-}max}$= 2990 < 4010		$P_{n\text{-}max}$= 2905 < 4010		$P_{n\text{-}max}$= 3466 < 4010		$P_{n\text{-}max}$= 3721 < 4010		$P_{n\text{-}max}$= 3540 < 4010	
	Pull-out	$P_{n\text{-}min}$= -441 > -1678		$P_{n\text{-}min}$= -738 > -1678		$P_{n\text{-}min}$= -1299 > -1678		$P_{n\text{-}min}$= -1422 > -1678		$P_{n\text{-}min}$= -1606 > -1678	
Cost (Php)	Pier	1,681,279	1.00	1,954,810	1.16	2,055,204	1.22	2,203,966	1.31	2,882,197	1.71
	Pile	2,100,000	1.00	2,520,000	1.20	2,520,000	1.20	2,520,000	1.20	3,360,000	1.60
	Total	3,781,279	1.00	4,474,810	1.18	4,575,204	1.21	4,723,966	1.25	6,242,197	1.65

1) Verification formula: △*Pu = 0.25*φ*Mn

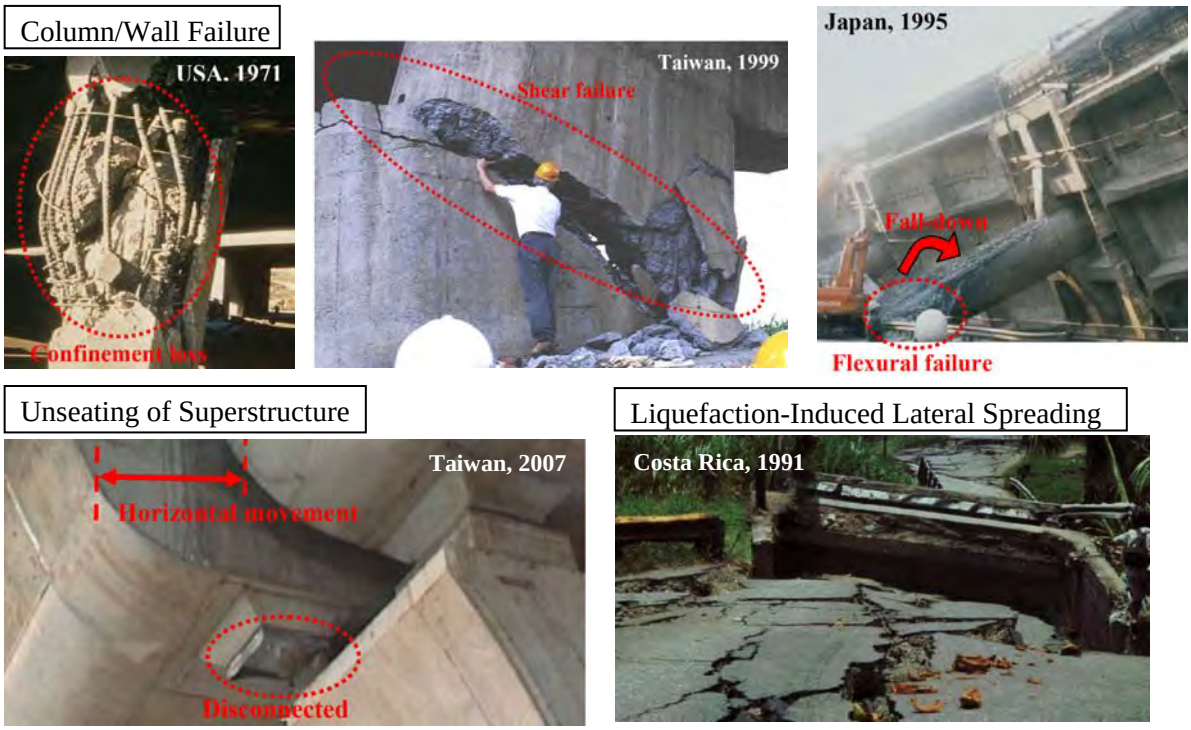
10.5 Policy and Outline of Example for Practical Application of Seismic Retrofit

In order to assist the design engineers with appropriate application of seismic retrofit schemes to existing structures, seismic retrofit work example is developed as an accompanying volume of the BSDS. The design example covers;

- seismic lessons learned from past earthquakes,
- outline of seismic retrofit schemes, and
- detail of each seismic retrofit scheme.

10.5.1 Seismic Lessons Learned from Past Earthquakes

Typical structural failures and summary of seismic vulnerability of old bridges are explained as follows.



(Source: Sixth National Seismic Conference, 2008)

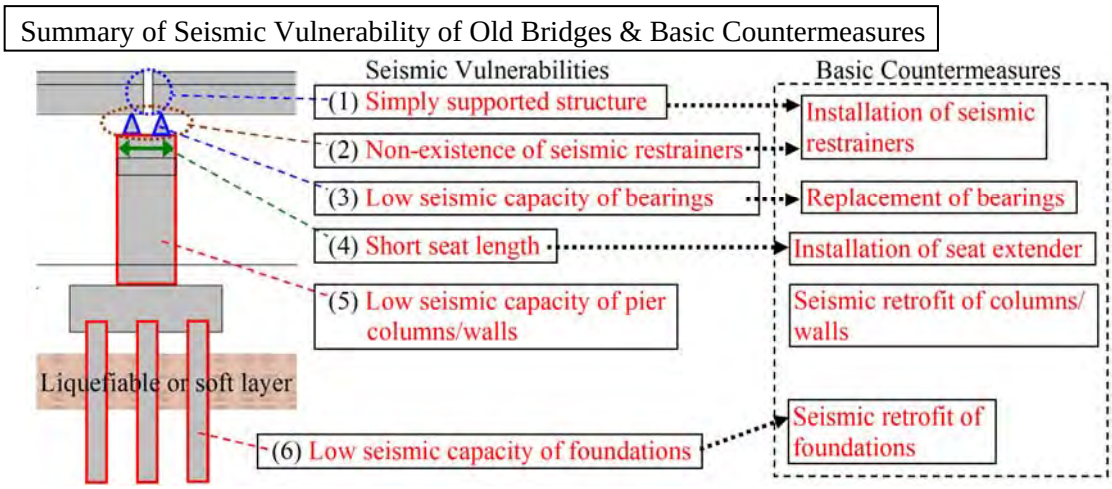


Figure 10.5.1-1 Typical Structural Failures Learned from Past Earthquakes

10.5.2 Outline of Seismic Retrofit Schemes

Basic concept of seismic retrofit planning is explained below for piers on land and piers in water, respectively.

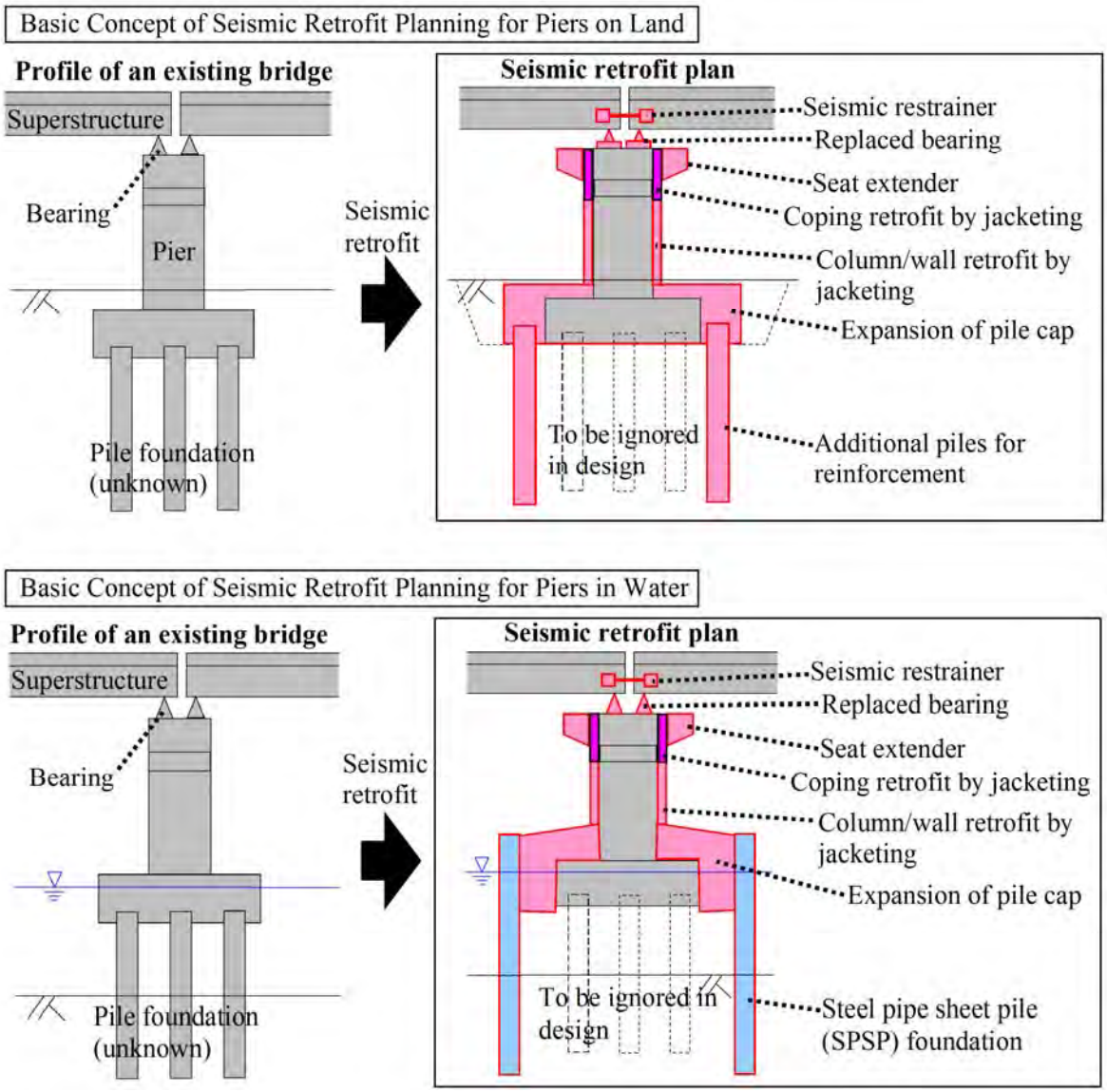


Figure 10.5.2-1 Basic Concept of Seismic Retrofit Planning

In addition to the basic seismic retrofit schemes explained above, the following three seismic retrofit schemes are introduced as additional options.

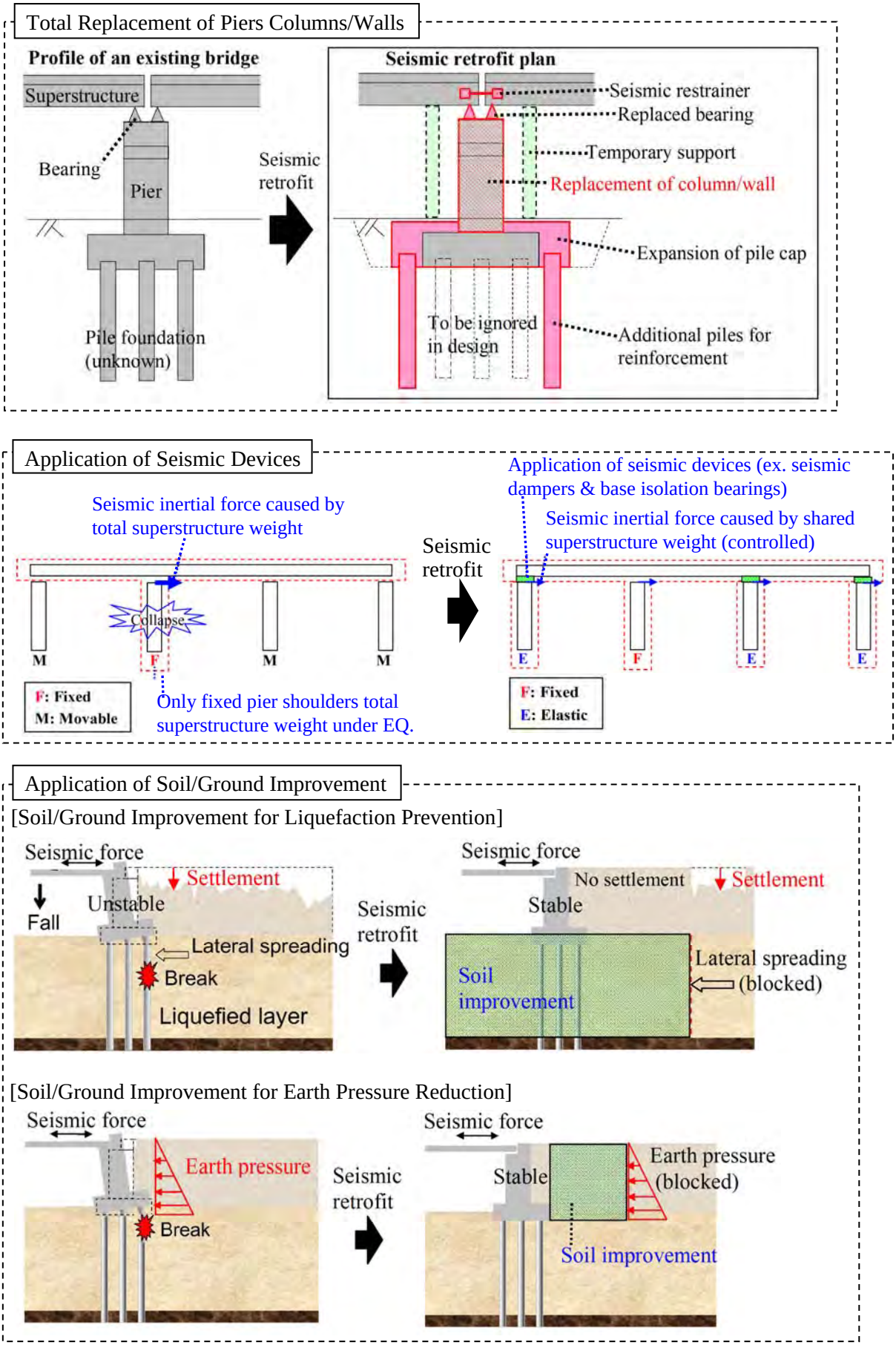


Figure 10.5.2-2 Additional Options for Seismic Retrofit Planning

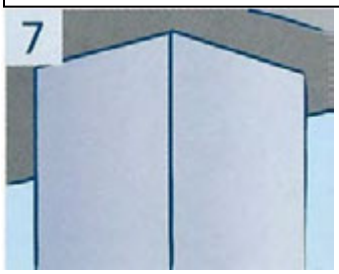
10.5.3 Detail of Each Seismic Retrofit Scheme

Detail (methodology or construction steps) of the following seismic retrofit schemes is explained with pictures of actual constructions.

Concrete Jacketing

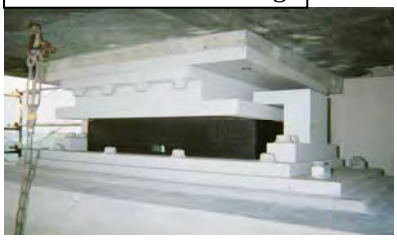
Steel Plate Jacketing

Carbon Fiber Sheet Jacketing



PC Panel Jacketing

Base Isolation Bearing



Seismic Damper
(Hydraulic Cylinder Type)

Unseating Prevention Cable/Belt/Chain/Stopper

Belt type

Cable type

Chain type

Stopper type

Seat Extender

Additional Piles (CCP)

Additional Piles (SPP)

SPSP Foundation

Soil/Ground Improvement

Source:
- Japan Bridge Association,
- Japan Bridge Bearing Association
- NETIS
- Source: Japanese Association for Steel Pipe Piles
- OCAJI

Figure 10.5.3-1 Detail of Each Seismic Retrofit Scheme