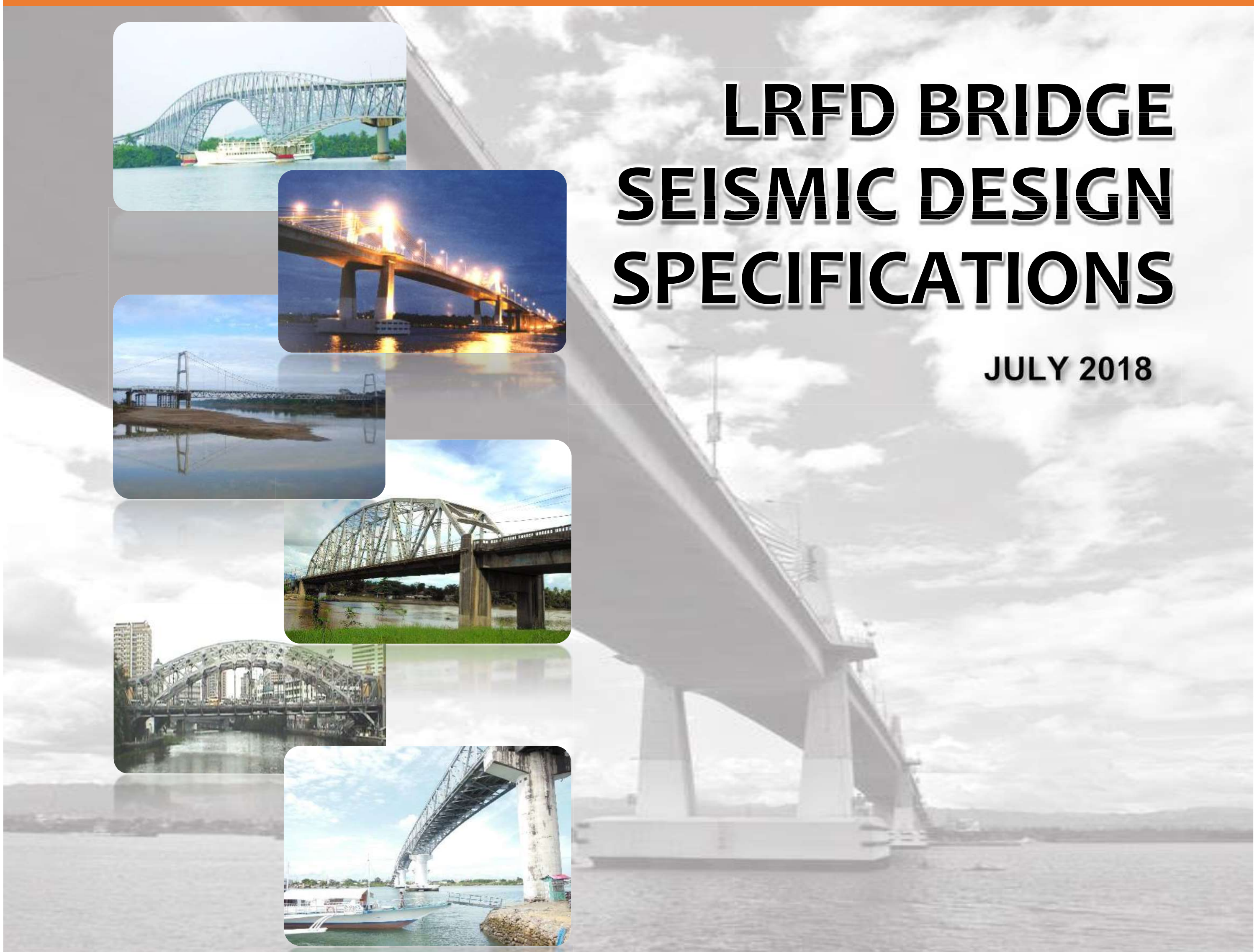
Republic of the Philippines
Department of Public Works and Highways

DPWH Guide Specifications



LRFD BRIDGE SEISMIC DESIGN SPECIFICATIONS

JULY 2018

2018 Interim Revisions

Republic of the Philippines
Department of Public Works and Highways

The DPWH Guide Specifications for LRFD Bridge Seismic Design Specifications
is prepared under the
Consulting Services for the Detailed Design and
Tender Assistance of the Metro Manila Priority
Bridges Seismic Improvement Project (MMPBSIP)
JICA Loan No. PH-P260 (Rebidding)

as implemented by: *in association with:*

PREFACE

The DPWH LRFD Bridge Seismic Design Specifications (BSDS), 2013 edition, was issued to provide guidance that will improve the seismic performance of bridges in the Philippines. However, many references were given to the AASHTO Specification prior to the publication of the DPWH Design Guidelines, Criteria & Standards (DGCS 2015). With the publication of the DGCS references were revised to follow the provisions of the DGCS.

Notable revisions to the BSDS includes:

- The use of a defined standard acceleration response spectra for Level 1 earthquake ground motion instead of the Level 1 acceleration coefficient contour maps, (3.4.1)
- Clarification on the definition of the soil N-values to the corrected N-values, (3.5.1,5.4.3.3),
- Coefficient of subgrade reaction equation, (4.4.2)
- Equation for P - Δ requirement (4.7)
- Expansion joint (5.9)
- Seat length (2.1) and excessive displacement (7.5)
- Seismically isolated bridges (8.2)
- Corrections of inadvertently typing errors, updates addition/deletion, and revisions.

This Interim Revision (2019) is part of the efforts of the DPWH to periodically improve the BSDS provisions as refinements and technical improvements become available.

Acknowledgment is given to the Japan International Cooperation Agency (JICA) for the support in the preparation of this manual.

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INSTRUCTIONS AND INFORMATION

GENERAL

This edition contains the Interim Revisions to the DPWH *LRFD Bridge Seismic Design Specifications (BSDS), 1st Edition (2013)*. The pages are intended to provide reference to the revised sections of BSDS.

AFFECTED SECTIONS AND CLAUSES

Underlined texts indicate the proposed revisions and approved in 2018 by the DPWH Bureau of Design (BOD). ~~Strikethrough texts~~ indicate proposed deletions likewise approved by the BOD.

All interim pages have section headings and the interim publication year. Main clauses and commentaries affected, revised, deleted or inserted are given in the interim pages indicating the particular items affected by this interim revision. Sections that are not affected under this 2018 interim revision are not included herein and is taken to remain in the original 2013 edition. The list of affected sections are given below.

List of Changed Articles:

SECTION 1 : INTRODUCTION

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SECTION 1:
INTRODUCTION

1.1 Background

Revise Clause (1) Paragraph 1 and 3 as follows:

(1) Prior to the development of the revision of the "*DPWH Design Guidelines, Criteria & Standards (2015)*" the ~~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.

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

Revise Clause (4) as follows:

(4) The ~~current~~ efforts of the DPWH to address the issues of advancement in engineering technology included the project "*Enhancement of Management and Technical Processes for Engineering Design in the DPWH*". Notable key component of the project is the updating and revision of the existing DPWH Guidelines and the standard drawings for GeoHazard Assessment (Vol. 2A), Engineering Surveys (Vol. 2B), Geological and Geotechnical Investigation (Vol. 2C), Water Engineering Projects (Vol. 3), Highway Design (Vol. 4), Bridge Design (Vol. 5), and Public Buildings and Other Related Structures (Vol. 6). ~~Surveys and Site Investigation (Vol. 1), Flood Control and Drainage Design (Vol. 3), Highways Design and Bridge Design (Vol. 4)~~. ~~The development of *Volume 4 – Bridge Design* covers bridge architecture, steel and concrete bridges, long span bridges, tunnels, bridge hydraulics, retrofitting of existing bridges and performance-based design, geo-hazard management, environmental safeguard, etc~~. Volume 5: Bridge Design applies to the design for construction, alteration, repair and retrofitting of highway bridges and other highway related structures but does not include provisions for bridges for railway, railway transit and public utilities. It was developed to update the 1984 DPWH bridge design guidelines and incorporated changes in the AASHTO code and other references. Volume 5 is divided into five parts: general information regarding bridges, structural design, structural design for seismic retrofitting, tunnels and sample calculations and drawings. However, although *Volume ~~4~~* 5 will cover most aspects of

bridge design, the *Bridge Seismic Design Specifications (BSDS)*, developed under ~~this~~ the JICA project, will be used as the section for seismic design provisions.

Revise Clause (5) as follows:

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

1.3 Scope of Specifications

Revise Clauses (1), (3), (4), (6) and (8) of this Section as follow:

(1) The scope of these Specifications covers mainly seismic design of bridges under the "Extreme Event Limit State for Earthquake Loading (Extreme Event 1)" following the force-based R-factor (response modification factor) design concept and philosophy of the *DPWH Design Guidelines, Criteria & Standards (2015)* and the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

~~Prior to the completion of the update and revision of the *DPWH Guide Specifications*, the~~ The design requirements for other limit states shall be that of the *DPWH Design Guidelines, Criteria & Standards (2015)*. Reference can be made to the *AASHTO LRFD Bridge Design Specifications*.

(3) The analysis and design provisions employed by the *DPWH Design Guidelines, Criteria & Standards (2015)* ~~*AASHTO Load and Resistance Factor Design (LRFD)*~~ methodology shall be adopted unless stated explicitly in these Specifications. When necessary, reference can be made to the *AASHTO Load and Resistance Factor Design (LRFD)*. Bridges shall be designed for the specified limit states to achieve the objectives of constructability, safety and serviceability with due regard to issues of inspectability, economy, and aesthetics.

(4) The applicability of these Specifications for other provisions to the types of new bridges with regards to conventional structural form and construction method shall be as specified in the *DPWH Design Guidelines, Criteria & Standards (2015)* ~~*AASHTO LRFD Specifications*~~. For non-conventional bridges and other types of construction (e.g. suspension bridges, cable stayed bridges, arch type bridges, and movable bridges), appropriate provisions of these Specifications may be adopted subject to prior approval by the DPWH.

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

(8) Other provisions not contained in these Specifications shall be referred to the *DPWH Design Guidelines, Criteria & Standards (2015)* ~~*AASHTO LRFD Bridge Design Specifications*~~. Further, reference is also made to the *AASHTO LRFD Bridge Design Specifications*, *Japan Road Association (JRA) Specifications for Highway Bridges Part V – Seismic Design* and *Part IV – Substructures* and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

1.4 SEISMIC DESIGN PHILOSOPHY

Revise Clause (5) – 1) as follows:

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

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

Commentary C1.4

Revise Commentary (6) as follows:

(6) Earthquake loads are given by the product of the elastic seismic response coefficient C_{sm} and the equivalent weight of the superstructure. The equivalent weight is a function of the actual weight and bridge configuration and is automatically included in both the single-mode and multimode methods of analysis specified in Article 4.1 to 4.3. Design and detailing provisions for bridges to minimize their susceptibility to damage from earthquakes ~~are contained in~~ shall comply with the *DPWH Design Guidelines, Criteria & Standards (2015)* or Sections 3, 4, 5, 6, 7, 10, and 11 of the *AASHTO LRFD Bridge Design Specifications 2012 (or later versions)*. ~~However, the design and detailing of members and components shall also comply with the requirements of the updated DPWH Guidelines, once completed~~. A flow chart summarizing these provisions is presented in Article 1.6.

1.5 LOADS AND LOAD AND RESISTANCE FACTORS FOR SEISMIC DESIGN

Revise Clause (1), (4) and (5) as follows:

(1) The combination of factored extreme force effects for Extreme Event I load combination, as provided in Section 10: Design Objectives, Loads and Load Factors of the *DPWH Design Guidelines, Criteria & Standards (2015)* shall be adopted for seismic design. Reference can also be made to Article 3.4 of the *AASHTO LRFD Bridge Design Specifications (2012)*. ~~, shall be adopted for seismic design in lieu of the on-going update and revision of the DPWH Guidelines (LRFD). Once completed, the load combination shall comply with the requirements of the DPWH Guidelines.~~

(4) The *DPWH Design Guidelines, Criteria & Standards (2015)* ~~*AASHTO LRFD Bridge Design Specifications*~~ provisions shall apply to both load and resistance factors, unless otherwise provided in these Specifications.

(5) The load factors for permanent loads, γ_p shall be that given in Table 10.3-2 of the *DPWH Design Guidelines, Criteria & Standards (2015)*. Reference can also be made to Table 3.4.1-2 and 3.4.1-3 of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*.

The load factor for live load effects, γ_{EQ} shall ~~be 0.50~~ be $\gamma_{EQ} = 0$ and $\gamma_{EQ} = 0.50$, in consideration of the live load force effect that will be disadvantageous to the bridge.

Commentary C1.5

Revise Commentary (1) and (2) as follows:

(1) These Specifications refers to Section 10: Design Objectives, Loads and Load Factors of the *DPWH Design Guidelines, Criteria & Standards (2015)* or Article 3.4 of the *AASHTO LRFD Bridge Design Specifications (version 2012)* that enumerates the loads and load combinations and load resistance factors for Extreme Event I relating to earthquake force effects. ~~Since the DPWH is currently updating and revising its design guidelines for bridges and other structures, the AASHTO LRFD loads and load and resistance factors are used in these Specifications. However, when the DPWH Guidelines are completed, the load combinations and load and resistance factors specified in the DPWH Guidelines shall be used.~~

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

Although the ~~The~~ use of $\gamma_{EQ} = 0.50$ is consistent with both the DPWH and AASHTO requirements to consider the possibility of live load presence during large earthquakes, especially in urban areas, the application of $\gamma_{EQ} = 0$ shall also be verified.

The use of the factor $\gamma_{EQ} = 0.50$ may result to large bending moment demands in the pier columns during earthquakes. However, the overstrength moment capacity (plastic) of the pier columns resulting from taking into consideration the axial forces with live load (γ_{EQ} = 0.50) could be less than that when live load effects are not considered (γ_{EQ} = 0). In this case, it is necessary to check the effects of the live load (with or without live load considerations) when determining the overstrength capacities of the columns and when designing the foundations.

1.6 SEISMIC DESIGN FLOWCHARTS

Revise Figure 1.6-2as follows:

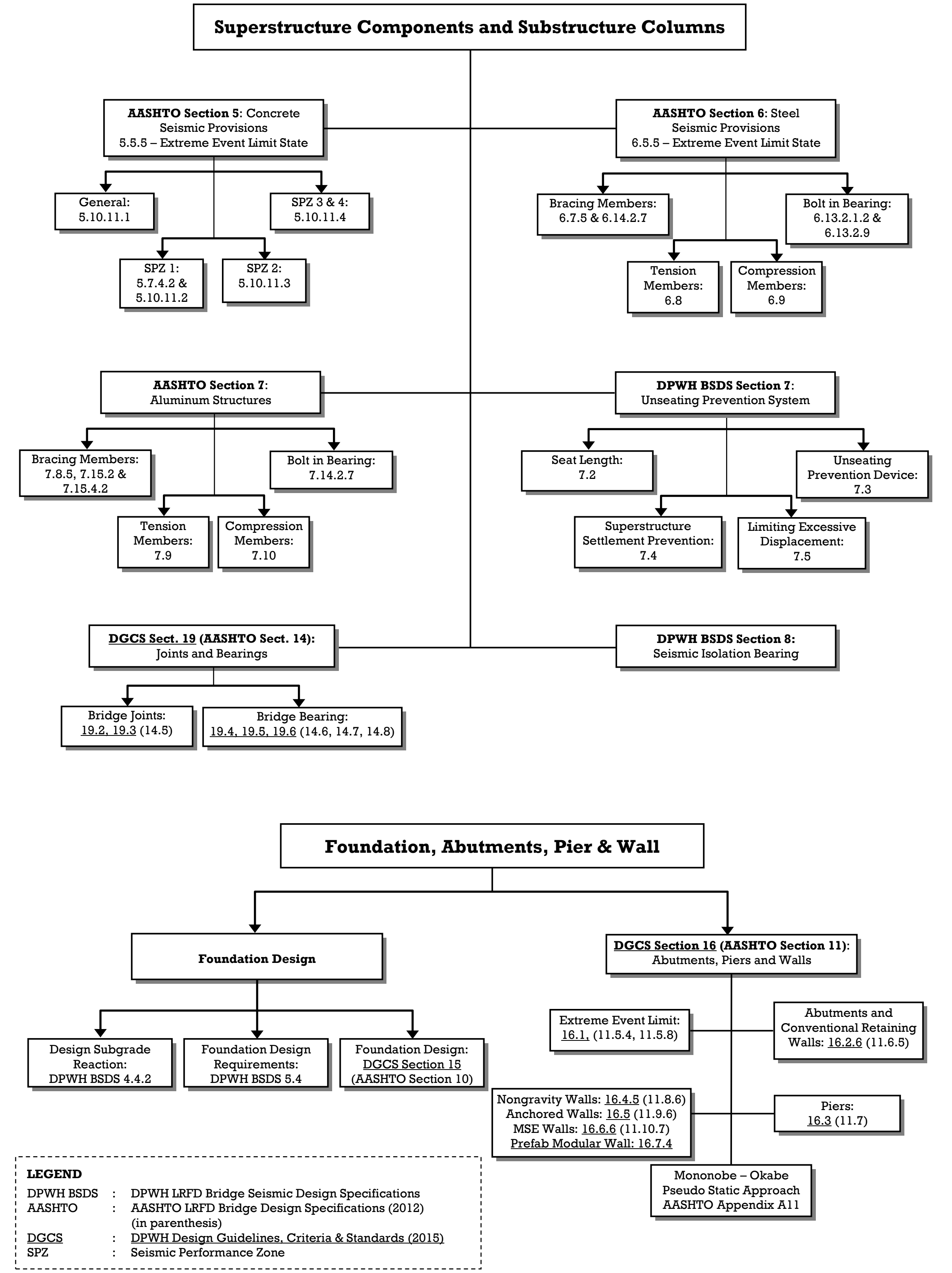

Figure 1.6-2 Seismic Detailing and Foundation Design Flow Chart

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SECTION 2:
DEFINITIONS AND NOTATIONS

2.2 NOTATIONS

Insert the following notations:

c_z : zone modification factor for Level 1 earthquake ground mMotion (EGM). The factor c_z varies with the one (1.0) second period horizontal response spectral acceleration coefficients (S_1) for Level 2 EGM.

c_D : modification factor for damping ratio.

N_{60} : SPT blow counts corrected for hammer efficiency

S : acceleration response spectra for Level 1 earthquake ground motion

S_o : standard acceleration response spectra (g) for Level 1 earthquake ground motion in accordance with the ground surface and the natural period T.

$S_{E\theta R}$: required girder seat length of a bridge, (m)

α_E : marginal unseating rotation angle, (degree); α_E can generally be taken as 2.5 degrees

α_2 : Safety factor used for calculation of the allowable ductility ratio of the reinforced concrete columns for Seismic Performance Level 2 or 3, and is specified as 1.2 in general.

θ' : skew angle used for evaluating the condition for rotation of a curved bridge, (degree)

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SECTION 3:
GENERAL REQUIREMENTS

3.1 APPLICABILITY OF SPECIFICATIONS

Revise Clause (6) as follows:

(6) Other provisions not contained in these Specifications shall conform to requirements of the *DPWH Design Guidelines, Criteria & Standards (2015)* or the *AASHTO LRFD Bridge Design Specifications, 2012 or later edition*. ~~However, the seismic design of bridges shall also conform to the requirements of the updated and revised DPWH Guide Specifications.~~

Commentary C3.1

Revise the second paragraph of Commentary as follows:

~~Since the DPWH is in the process of updating and revising its Design Guide Specifications, these~~ These Specifications shall comply with the other provisions and requirements of the *DPWH Design Guidelines, Criteria & Standards (2015)* or the *AASHTO LRFD Bridge Design Specifications, 2012 or later edition* for specific requirements not contained herein. ~~However, once the DPWH Design Guide Specifications have been completed, these Specifications shall be use together with the DPWH Design Guide Specifications.~~

3.3 SEISMIC PERFORMANCE REQUIREMENTS

3.3.1 General

Insert Level 1 Design Acceleration Response Spectra in Table 3.3.1-1 as follows:

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

Earthquake Ground Motion (EGM)	**Bridge Operational Classification**		
	OC-I (Critical Bridges)	**OC-II (Essential Bridges)**	**OC-III (Other Bridges)**
Level 1 (Small to moderate earthquakes which are highly probable during the bridge service life, 100-year return. The Design Acceleration Response Spectra in accordance with the provisions of Section 3.4.1(5) and Table 3.4.1-1 or the provisions of Section 3.4.2 for Site Specific Procedure)	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. The Design Acceleration Response Spectra in accordance with the provisions of Section 3.4.1 for the General Procedure or the provisions of Section 3.4.2 for Site Specific Procedure)	SPL-2 *(Limited seismic damage and capable of immediately recovering bridge functions without structural repair)*	SPL-2 *(Limited seismic damage and capable of recovering bridge function with structural repair within short period)*	SPL-3 *(May suffer damage but should not cause collapse of bridge or any of its structural elements)*

3.4 SEISMIC HAZARD

3.4.1 General Procedure

Revise Clause (1) as follows:

(1) The General Procedure for Level 2 Earthquake Ground Motion shall use the peak ground acceleration coefficient (*PGA*) and the short- and long-period spectral acceleration coefficients (S_S and S_1 respectively) to calculate the design response spectrum as specified in Article 3.6.

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

Insert Clause (4), Equation 3.4.1-1, Equation 3.4.1-2, Table 3.4.1-1 and Table 3.4.1-2 as follows:

(4) The General Procedure to determine Level 1 Earthquake Ground Motion shall be in accordance with the acceleration response spectra provided at the ground surface prescribed in Section 3.5.2 and calculated by Equation 3.4.1-1.

$$S = c_z c_D S_0 \qquad (3.4.1\text{-}1)$$

where:

S : Acceleration Response Spectra for Level 1 Earthquake Ground Motion.

c_z : Zone Modification factor for Level 1 Earthquake Ground Motion (EGM) in accordance with Table 3.4.1-1. The factor c_z varies with the one (1.0) second period horizontal response spectral acceleration coefficients (S_1) for Level 2 EGM prescribed in Section 3.6 and given in Appendix 3B (Figures 3B-37 to 3B-54).

Table 3.4.1-1 Zone Modification Factor for Level 1 Earthquake Ground Motion

1.0-sec Horizontal Acceleration Coefficient (S_1) for Level 2 EGM	Zone Modification Factor
S_1 ∗ 0.25	0.70
$0.25 < S_1$ ∗ 0.35	0.85
$S_1 > 0.35$	1.0

c_D : Modification factor for damping ratio, calculated by Equation 3.4.1-2 in accordance with the damping ratio h.

$$c_D = \frac{1.5}{40h + 1} + 0.5 \qquad (3.4.1\text{-}2)$$

S_0 : Standard Acceleration Response Spectra (g) for Level 1 Earthquake Ground Motion given in Table 3.4.1-2 in accordance with the ground surface prescribed in Section 3.5.2 and the natural period T.

Table 3.4.1-2 Standard Acceleration Response Spectra (S_0) for Level 1 Earthquake Ground Motion

Ground Type	**S_0 (g) with Natural Period T (s)**		
Ground Type I	T < 0.1 $S_0 = 0.439\ T^{1/3}$ but S_a ≧ 0.16	0.1 ∗ T ∗ 1.1 $S_0 = 0.204$	1.1 < T $S_0 = 0.224/T$
Ground Type II	T < 0.2 $S_0 = 0.435\ T^{1/3}$ but S_a ≧ 0.20	0.2 ∗ T ∗ 1.3 $S_0 = 0.255$	1.3 < T $S_0 = 0.331/T$
Ground Type III	T < 0.34 $S_0 = 0.438\ T^{1/3}$ but S_a ≧ 0.34	0.34 ∗ T ∗ 1.5 $S_0 = 0.306$	1.5 < T $S_0 = 0.459/T$

Commentary C3.4.1

Revise Paragraph 1 of Comment (1) as follows:

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

Revise Paragraph 3 of Comment (1) as follows:

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

Add Comment (3) and Figure C.3.4.1-1 as follows:

(3) Based on the comparison between the seismic hazard maps developed for the 100-year return period and the JRA-2002[1] Level 1 Earthquake which has been established from numerous actual recorded data, it is found that:

1. The Level 1 spectra determined from the spectral acceleration maps (Figure 3A-1 to 3A-18 for PGA; Figure 3A-19 to 3A-36 for S_S; Figure 3A-37 to 3A-54 for S_1) obtained using PSHA analyses seem to generate higher spectral acceleration values as compared to the Japanese experience.

   The study undertaken by Douglas and Edwards (2016)[2] stated that the use of currently-available GMPEs (ground motion prediction equations) derived mainly from using datasets of large events and functional forms made to capture large-event ground motion characteristics will naturally lead to over-prediction when used in PSHA analyses to establish small earthquake ground motion (in the present case, Level 1 Ground Motion). There had been little call in the past for GMPEs that could be used confidently for small earthquakes until the current evolvement into performance-based seismic design.

2. Moreover, the Site Factors in Table 3.5.3-1 for F_{pga}, Table 3.5.3-2 for F_a, and Table 3.5.3-3 for F_v are established for the Level 2 spectral acceleration values and are not appropriate for adjusting the lower ranges of Level 1 spectral acceleration values.

In this interim revision, it is recommended that the Level 1 Earthquake Ground Motion adapted from JRA 2012 will be taken as the ground motion of medium strength with high probability of occurrence given in Table 3.4.1-1 and plotted as shown in Figure C3.4.1-1. This JRA Level 1 Earthquake Ground Motion, established for each ground type defined in Section 3.5.1, is based on the results of statistical analysis of acceleration response spectra with 5% damping ratio obtained from strong motion ground records in Japan, considering the characteristics of past earthquake damage, vibration properties of the ground, and other engineering evaluations.

[1] The Level 1 spectra in JRA2002 had been established based on multiple regression analysis using 394 components of strong ground motion records.

[2] John Douglas and Benjamin Edwards, "Recent and future developments in earthquake ground motion estimation," Earth-Science Reviews, Vol. 160, pp. 203–219, 2016.

This Level 1 Earthquake Ground Motion will used in lieu of those obtained by Appendix 3A of the BSDS until such time that the PSHA procedures for estimating smaller-event earthquake ground motion level (GMPEs scaled for smaller earthquake in particular) are reliably available.

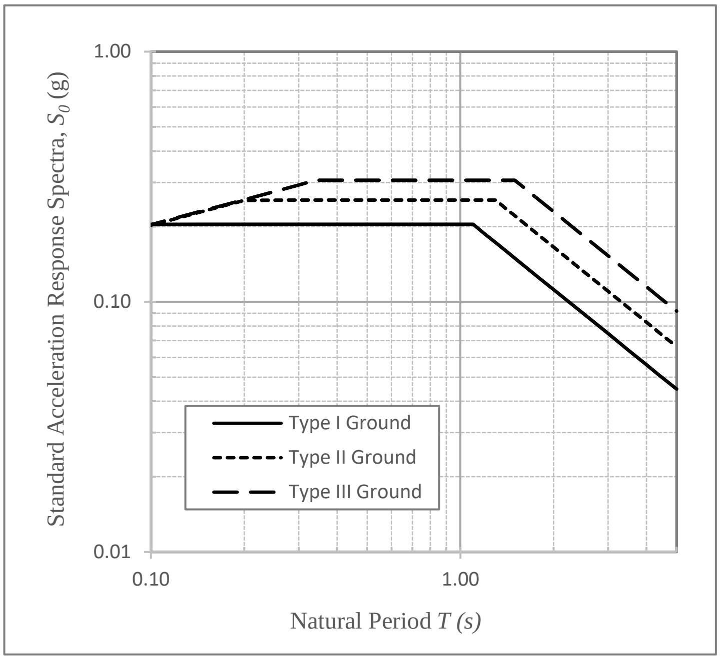

Figure C.3.4.1-1 Standard Acceleration Response Spectra So for Level 1 Earthquake Ground Motion

3.5 SITE EFFECTS

3.5.1 Ground Types Definitions (Site Class Definitions)

Commentary C3.5.1

Add Equation C3.5.1-2 ($N = \frac{1}{1.2}N_{60}$), revise the notation N, revise the third paragraph and insert the next paragraph to this commentary as follows:

The values of N shall be corrected to N60 and divided by a factor of 1.2 as follows:

$$N_i = \frac{1}{1.2} N_{60i} \quad \ldots\ldots \quad \text{(C3.5.1-2)}$$

where:

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

N_{60} : SPT blow counts corrected for hammer efficiency

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

Previous studies[3,4] indicated that the results of the standard penetration test varies, depending on the hammer efficiency, borehole diameter, sampler type, rod length, etc. References [3] and [4] compared the results of the different methods of performing SPT tests in difference countries (including Japan) with different equipment. Since the SPT hammer types and efficiencies used in the Philippines varies with that in Japan, resulting to different N-values[5], the N-values obtained from the geotechnical exploration shall be corrected to N_{60} and divided by a factor of 1.2.

3.5.3 Site Factors

Revise the paragraph as follows:

The Site Factors F_{pga}, F_a and F_v specified in Tables 3.5.3-1, 3.5.3-2, and 3.5.3-3 shall be used in the zero-period, short-period range, and long-period range, respectively for the Level 2 elastic seismic response coefficient in the design response spectrum of Article 3.6 of this Section. These factors shall be determined using the Ground Types (Site Class) given in Table 3.5.1-1 and the mapped values of the coefficients *PGA*, S_S, and S_1 in Figures ~~3.4.1-1~~ 3.4.1-4 to 3.4.1-6 and Appendix ~~3A and~~ 3B.

[3] H.B. Seed, K. Tokimatsu, L.F. Harder and R.M. Chung, "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", ASCE Journal of Geotechnical Engineering, Vol. 111, No.12, Dec. 1985, pp. 1425-1445.

[4] A.W. Skempton, "Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, relative Density, Particle Size, Ageing and Overconsolidation", Geotechnique 36, No.3, pp. 425-447.

[5] K. Tokimatsu, H. Kojima, S. Kuwayama, A. Abe, and S. Midorikawa, "Liquefaction-Induced Damage to Buildings in 1990 Luzon Earthquake", ASCE Journal of Geotechnical Engineering, Vol. 120, No. 2, Feb. 1994, pp. 290-307.

3.8 Response Modification Factors

3.8.1 General

Revise clause (1) as follows:

(1) To apply the response modification factors specified herein, the structural details shall satisfy the provisions of Articles ~~5.10.2.2~~ 12.7.2.2 (Concrete Structures, Seismic Hooks – Details of Reinforcement), ~~5.10.11~~ 12.7.11 (Concrete Structures, Provisions for Seismic Design – Details of Reinforcement), and ~~5.13.4.6~~ 12.10.4.5 (Concrete Structures, Seismic Requirements – Specific Members) of the DPWH Design Guidelines, Criteria and Standards (2015) or reference can be made with AASHTO LRFD Bridge Design Specifications (2012 or later). ~~The structural details shall likewise comply with the revised and updated DPWH Guide Specifications.~~

APPENDIX 3A:
SPECTRAL ACCELERATION MAPS FOR LEVEL 1 EARTHQUAKE GROUND MOTION
(100-YEAR RETURN PERIOD)

Delete the following paragraph:

~~***Appendix 3A*** presents the spectral acceleration maps for Level 1 Earthquake Ground Motion (100-year return period) consisting of:~~

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

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

Add the following paragraph to the end of this Section:

Since Appendix 3A was developed following the procedures of PSHA using GMPEs (ground motion prediction equations) derived mainly from using datasets of large events and functional forms made to capture large-event ground motion characteristics, the resulting Level 1 ground motions tend to be overestimated. Moreover, the Site Factors in Table 3.5.3-1 for F_{pga}, Table 3.5.3-2 for F_a, and Table 3.5.3-3 for F_v are established for the Level 2 spectral acceleration values and are not appropriate for adjusting the lower ranges of Level 1 spectral acceleration values.

Until such time that PSHA procedures for estimating smaller-event earthquake ground motion level (GMPEs scaled for smaller earthquake in particular) are reliably available, the spectral acceleration maps given in Appendix 3A shall not be used for Level 1 earthquake ground motions. Instead, the General Procedure to determine Level 1 Earthquake Ground Motion will be in accordance with the acceleration response spectra provided at the ground surface prescribed in Section 3.4.1 and given in Table 3.4.1-1.

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APPENDIX 3B:
SPECTRAL ACCELERATION MAPS FOR LEVEL 1 EARTHQUAKE GROUND MOTION
(1,000-YEAR RETURN PERIOD)

Revise Level 1 with Level 2 in the following figure titles:

Figure 3B-1 Horizontal Peak Ground Acceleration Coefficient (*PGA*) for Level ~~1~~2 Earthquake Ground Motion (Entire Country)

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

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

Figure 3B-4 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (CAR)

Figure 3B-5 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region III)

Figure 3B-6 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (NCR)

Figure 3B-7 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region IV-A)

Figure 3B-8 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region V)

Figure 3B-9 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region IV-B)

Figure 3B-10 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region VI)

Figure 3B-11 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region VII)

Figure 3B-12 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region VIII)

Figure 3B-13 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region IX)

Figure 3B-14 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region X)

Figure 3B-15 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region XI)

Figure 3B-16 Horizontal Peak Ground Acceleration Coefficient (PGA) for Level ~~1~~2 Earthquake Ground Motion (Region XIII)

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SECTION 4: ANALYSIS REQUIREMENTS

4.1 GENERAL

Revise Clause (1) as follows:

(1) This section describes the analysis requirements for seismic effects on bridges under the extreme event limit state. Other analysis requirements not specified in this Section shall comply with the requirements of *Section 11 (Structural Analysis and Evaluation)* of the *DPWH Design Guidelines, Criteria & Standards (2015)*, or alternatively *Section 4: (Structural Analysis and Evaluation), of the AASHTO LRFD Bridge Design Specifications (2012 or later)* concerning the methods of modeling and analysis for the design and evaluation of bridges. ~~Likewise, the analysis for bridges shall be consistent with the updated and revised DPWH Guide Specifications.~~

4.2 Single-Span Bridges

Revise Clause (1) as follows:

(1) The seismic analysis and design for abutments of single-span bridges shall comply with the requirements of *Section 16 (Walls, Abutments and Piers)* of the *DPWH Design Guidelines, Criteria & Standards (2015)*, or alternatively *Section 11 (Walls, Abutments and Piers) of the AASHTO LRFD Bridge Design Specifications 2012 (or later versions)*.

4.3 Multispan Bridges

4.3.3 Multimode Spectral Method

Revise Clause (1) as follows:

(1) The multimode spectral analysis method shall be used for bridges in which other modes, in addition to the fundamental mode, participate significantly in the response of the bridge. These modes may be in the same coordinate direction or have coupled response in two or three directions. ~~coupling occurs in more than one of the three coordinate directions within each mode of vibration.~~ As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

4.3.4 Time-History Method

4.3.4.1 General

Revise Clause (2) as follows:

(2) Any step-by-step time-history method of analysis used for either elastic or inelastic analysis shall satisfy the requirements of *Section 11.4* of the *DPWH Design Guidelines, Criteria & Standards (2015)*, or alternatively *Article 4.7 (Dynamic Analysis) of AASHTO LRFD Bridge Design Specifications (2012 or later)*.

4.3.4.2 Acceleration Time Histories

Commentary C4.3.4.2

Revise Commentary Clause (2) as follows:

(2) Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps or PHIVOLCS data base, shall be used. ~~Reference can also be made from deaggregation information on the USGS website: *http://geohazards.cr.usgs.gov*.~~ (Deaggregation information for major bridge sites in the Philippines are currently not available online).

4.4 Mathematical Model

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

Commentary C4.4.2

Revise Equation C4.4.2-4 of Commentary Clause (3)-2) as follows:

$$k_H = k_{HO}\left(\frac{\mathrm{B_H}}{0.3}\right)^{-3/4} \qquad \text{(C.4.4.2-4)}$$

Revised row 3 of Table C.4.4.2-2 as follows:

TableC.4.4.2-2 Equivalent Loading Width of Foundation B_H

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

4.7 *P-Δ* Requirements

Delete the factor 12 in Equation 4.7-2 as follows:

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

SECTION 5:
DESIGN REQUIREMENTS

5.1 GENERAL

Revise Clause (1) as follows:

(1) This Section describes the design requirements for earthquake effects on bridges under the extreme event limit state. For other design and detail requirements not specified in this Section the design shall comply with the requirements of the *DPWH Design Guidelines, Criteria & Standards (2015)* or alternatively the *AASHTO LRFD Bridge Design Specifications (2012 or later)* corresponding to the components, materials and construction provisions, where applicable. ~~The design shall also comply with the requirements of the updated and revised DPWH Guide Specifications.~~

Commentary C5.1

Revise Commentary as follows:

The provisions covered in this Section focuses on the design requirements for extreme event limit state under earthquake effects. The design of members not covered under this Section and the design for other limit states shall comply with the *DPWH Design Guidelines, Criteria & Standards (2015)* and when necessary, the *AASHTO LRFD Bridge Design Specifications (2012 or later)*. ~~However, since the DPWH is in the process of updating and revising the design guidelines and specifications, the requirements for design shall also be consistent with the updated and revised DPWH Design Guidelines.~~

5.3 CALCULATION OF DESIGN FORCES

5.3.1 General

Commentary C5.3.1

Revise Clause (1) Commentary as follows:

(1) This Article refers to superstructure effects carried into the substructure. Abutments on multi-span bridges and retaining walls are subject to acceleration-augmented soil pressures as specified in *Sections 10.15.3 and 16.2.6 of the DPWH Design Guidelines, Criteria and Standards (2015)* or *Articles 3.11.4 and 11.6.5 of the AASHTO LRFD Bridges design Specifications (2012)*. Although abutments and wingwalls on single-span structures are not fully covered at this time, the Engineer should use judgement in this area. Reference shall be made to *Section 16 of the DPWH Design Guidelines Criteria and Standards (2015)* or *Section 11 (Walls, Abutments and Piers) of the AASHTO LRFD Bridge Design Specifications 2012(or later version).*

5.3.2 Seismic Performance Zone 1

Commentary C5.3.2

Revise Commentary (3) and (4) as follows:

(3) Lateral connection forces are transferred from the superstructure into the foundation elements through the substructure. The force effects in this load path from seismic and other lateral loads should be addressed in the design. If each bearing supporting a continuous segment or simply supported span is an elastomeric bearing, there may be no fully restrained directions due to the flexibility of the bearings. However, the forces transmitted through these bearings to substructure and foundation elements should be determined in accordance with this Article and where applicable, with Section 19.1 of the *DPWH Design Guidelines, Criteria & Standards (2015)* or *Article 14.6.3 of the AASHTO LRFD Bridge Design Specifications (2012)*.

(4) The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{EQ}, used in conjunction with Article 1.5 of these Specifications and Table 10.3-1 of the *DPWH Design Guidelines, Criteria & Standards (2015)* or Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications (2012)*.

5.3.3 Seismic Performance Zone 2

Revise Clause (4) as follows:

(4) Where a group load other than Extreme Event I, as specified in Table 10.3-1 of the *DPWH Design Guidelines, Criteria & Standards (2015)* or *Table 3.4.1-1 (Load Combinations and Load Factors)* of the *AASHTO LRFD Bridge Design Specifications (2012)*, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than those calculated using the procedure specified above, due to a possible overstrength of the columns, shall be considered.

5.3.4 Seismic Performance Zones 3 and 4

5.3.4.3 Inelastic Hinging Forces

5.3.4.3a General

Revise Clause (2) as follows:

(2) Inelastic hinges shall be ascertained to form before any other failure due to overstress or instability in the structure and/or in the foundation. Inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired. Inelastic flexural resistance of substructure components shall be determined in accordance with the provisions of *Sections 5 (Concrete Structures)* and *Section 6 (Steel Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)* or if applicable, the provisions of the *DPWH Design Guidelines, Criteria & Standards (2015)*.

5.3.4.3.d Column and Pile Bent Design Forces

Revise Bullet 1 as follows:

- Axial Forces – The maximum and minimum design forces determined using Extreme Event load Combination I with either the elastic design values determined in Articles ~~4.2.1~~ 5.2 taken as EQ, or the values corresponding to plastic hinging of the column taken as EQ.

5.3.4.3.e Pier Design Forces

Revise the Paragraph as follows:

The design forces shall be those determined for Extreme Event Limit State Load Combination I, except where the pier is designed as a column in its weak direction. If the pier is designed as a column, the design forces in the weak direction shall be as specified in Article 5.3.4.3d of this Section and all the design requirements for columns, as specified in Section 12 (Concrete Structures) of the *DPWH Design Guidelines, Criteria & Standards (2015)* or *Section 5 (Concrete Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*, shall apply. When the forces due to plastic hinging are used in the weak direction, the combination of forces, specified in Article 5.2 of this Section, shall be applied to determine the elastic moment which is then reduced by the appropriate *R*-factor.

Commentary C5.3.4.3e

Revise the Commentary as follows:

The design forces for piers specified in Article 5.3.4.3e of this Section are based on the assumption that a pier has low ductility capacity and no redundancy. As a result, a low *R*-factor of 2 is used in determining the reduced design forces, and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction, then both the design forces and, more importantly, the design requirements of Article 5.3.4.3d of this Section of these Specifications and Section 12 (Concrete Structures) of the *DPWH Design Guidelines, Criteria & Standards (2015)* or *Section 5 (Concrete Structures)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later)*, are applicable

5.4 FOUNDATION REQUIREMENTS

5.4.2 Spread Foundation

Revise Clause (1) as follows:

(1) The following provisions shall apply to foundation design for extreme event limit state under Level 2 Earthquake Ground Motion. For other limit states, the design of foundation shall comply with the requirements of *Section 15 (Foundations)* of the *DPWH Design Guidelines, Criteria & Standards (2015) or Section 10 (Foundations)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later),* where applicable

5.4.3 Pile Foundation

5.4.3.1 General

Revise Clause (1) as follows:

(1) The following provisions shall apply to foundation design for extreme event limit state under Level 2 Earthquake Ground Motion. For other limit states, the design of foundation shall comply with the requirements of *Section 15 (Foundations)* of the *DPWH Design Guidelines, Criteria & Standards (2015) or Section 10 (Foundations)* of the *AASHTO LRFD Bridge Design Specifications (2012 or later),* where applicable.

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

Commentary C5.4.3.3

Revise Clause (1)2) Paragraph 2, last sentence as follows:

In general, the factored bearing resistance capacity of a bridge foundation may be calculated by multiplying the nominal bearing capacity obtained from the load test (at the surrounding site) by 1.2 (modification coefficient for the nominal bearing capacity shown in Table ~~C~~5.4.3.3-1), if the following conditions are satisfied:

Insert call-out in Figure C5.4.3.3-1:

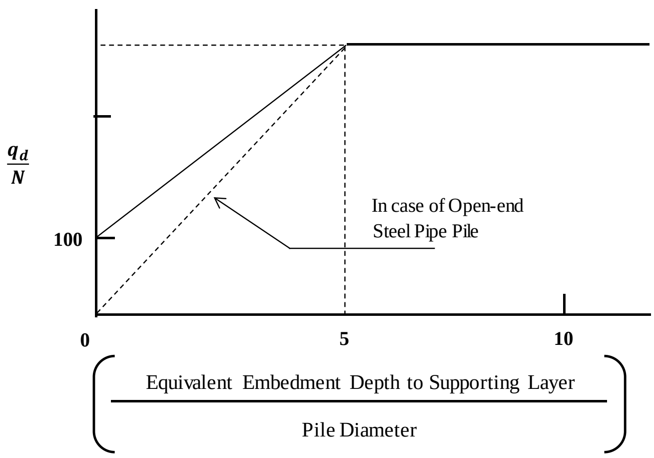


Figure C5.4.3.3-1 Evaluation Chart for Ultimate End Bearing Capacity Intensity *(q_d)*

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

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

Notes: q_u : unconfined compressive strength (kN/m^2)

N : $\frac{1}{1.2} N_{60}$ -values from the Standard Penetration Test (SPT)

N_{60} : SPT N blow counts corrected for hammer efficiency

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

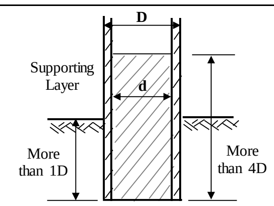
<table>
<tr><th>Pile Tip Treatment Method</th><th>Evaluation Methods of Nominal Bearing Resistance Intensity at Pile Tip</th></tr>
<tr><td>Final Driving Method</td><td>Evaluation Methods for Driven Piles can be applicable</td></tr>
<tr><td>Cement Milk
Jetting and Mixing Method</td><td>Nominal End Bearing Resistance Intensity (kN/m^2)
$q_d = \begin{cases} 150N\ (\leq 7{,}500), \text{ for Sandy Layer} \\ 200N\ (\leq 10{,}000), \text{ for Gravelly Layer} \end{cases}$
where:
N : $\frac{1}{1.2} N_{60}$ -values from the Standard Penetration Test (SPT)
N_{60} : SPT N blow counts corrected for hammer efficiency</td></tr>
<tr><td>Concrete
Placing
Method

</td><td>Nominal End Bearing Resistance Intensity of Cast-in-place Piles can be applicable</td></tr>
</table>

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

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

Note: N : $\frac{1}{1.2} N_{60}$ -values from the Standard Penetration Test (SPT)

N_{60} : SPT N blow counts corrected for hammer efficiency

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

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

Note: N : $\frac{1}{1.2}N_{60}$ -values from the Standard Penetration Test (SPT)

N_{60} : SPT N blow counts corrected for hammer efficiency

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

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

Note: c : cohesion of ground (kn/m^2),

N : $\frac{1}{1.2}N_{60}$ -values from the Standard Penetration Test (SPT)

N_{60} : SPT N blow counts corrected for hammer efficiency

5.4.3.7 Calculation of Pile Reaction and Displacements

Commentary C5.4.3.7

Revise numbering of the call-out in Figure 5.4.3.7-1

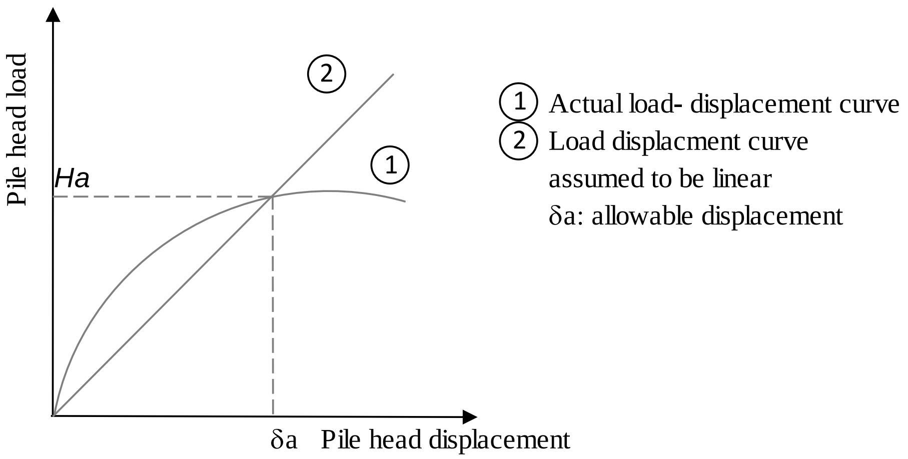


Figure 5.4.3.7-1

5.4.3.8 Design of Piles Against Loads After Construction

Commentary C5.4.3.8

Replace β_x *with* βx (β *multiplied by x) in Equation C5.4.3.8-2:*

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

Replace all β_x *with* βx (β *multiplied by x) in Table C5.4.3.8-1a:*

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

a	Deflection curve, y (mm)	$y=\frac{H}{2EI\beta^3}e^{-\beta x}[(1+\beta h_0)\cos\beta\ x - \beta h_0 \sin\beta x]$	$y=\frac{H}{2EI\beta^3}e^{-\beta x}\cos\beta x$	$y=\frac{H}{4EI\beta^3}e^{-\beta x}[\cos\beta x+\sin\beta x]$
e	Bending moment at each section of the pile, M (N-mm)	$M=-\frac{H}{\beta}e^{-\beta x}[\beta h_0\cos\beta x + (1+\beta h_0)\sin\beta x]$	$M=-\frac{H}{\beta}e^{-\beta x}\sin\beta x$	$M=-\frac{H}{2\beta}e^{-\beta x}(\sin\beta x-\cos\beta x)$
f	Shear force at each section of the pile, S (N)	$S=-He^{-\beta x}[\cos\beta x-(1+2\beta h_0)\sin\beta x]$	$S=-He^{-\beta_x}(\cos\beta x-\sin\beta x)$	$S=-He^{-\beta_x}\cos\beta x$

Replace all β_x *with* βx (β *multiplied by x) in Table C5.4.3.8-1b:*

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

a	Deflection curve, y (mm)	$y_1=\frac{H}{6EI\beta^3}[\beta^3x^3+3\beta^3(h+h_0)x^2 - 3\{1+2\beta(h+h_0)\}\beta x + 3\{1+\beta(h+h_0)\}]$ $y_2=\frac{H}{2EI\beta^3}e^{-\beta x}[\{1+\beta(h+h_0)\}\cos\beta x - \beta(h+h_0)\sin\beta x]$	$y_1=\frac{H}{6EI\beta^3}\{\beta^3x^3+3\beta^3hx^2$ - 3(1+2βh)β x + 3(1+βh)} $y_2=\frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h)\cos\beta x - \beta h\sin\beta x\}$	$y_1=\frac{H}{12EI\beta^3}[2\beta^3x^3-3(1-\beta h)\beta^2x^2 - 6\beta^2hx+3(1+\beta h)]$ $y_2=\frac{H}{4EI\beta^3}e^{-\beta x}[(1+\beta h)\cos\beta x + (1-\beta h)\sin\beta x]$
e	Bending moment at each section of the pile, M (N-mm)	$M_1=-H(x+h)-M_t = -H(x+h+h_0)$ $M_2=-\frac{H}{\beta}e^{-\beta x}[\beta(h+h_0)\cos\beta x + \{1+\beta(h+h_0)\}\sin\beta x]$	$M_1=-H(x+h)$ $M_2=-\frac{H}{\beta}e^{-\beta x}\{\beta h\cos\beta x + (1+\beta h)\sin\beta x\}$	$M_1=\frac{H}{2\beta}[-2\beta_x+(1-\beta h)]$ $M_2=\frac{H}{2\beta}e^{-\beta x}[(1-\beta h)\cos\beta x - (1+\beta h)\sin\beta x]$
f	Shear force at each section of the pile, S (N)	$S_1=-H$ $S_2=-He^{-\beta x}[\cos\beta x-\{1+2\beta\bullet(h+h_0)\}\sin\beta x]$	$S_1=-H$ $S_2=-He^{-\beta x}[\cos\beta x-(1+2\beta h)\sin\beta x]$	$S_1=-H$ $S_2=-He^{-\beta x}(\cos\beta x-\beta h\sin\beta x)$

Replace all β_x *with* βx *(*β *multiplied by x) in Table C5.4.3.8-2:*

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

a	Deflection curve, *y* (mm)	$y_1 = f - \frac{1}{2EI\beta^3}(-C_1 - C_2 + C_3 - C_4)x + \frac{M_t + Hh}{2EI}x^2 + \frac{H}{6EI}x^3$	$y_2 = \frac{1}{2EI\beta^3}\left[e^{-\beta x}(C_1 \cos\beta x + C_2 \sin\beta x) + e^{-\beta x}(C_3 \cos\beta x + C_4 \sin\beta x)\right]$
b	Bending moment at each section of the pile, *M* (N-mm)	$M_1 = -H(x+h) - M_t$	$M_2 = \frac{1}{\beta}\left[e^{\beta x}(C_1 \sin\beta x - C_2 \cos\beta x) + e^{-\beta x}(-C_3 \sin\beta x + C_4 \cos\beta x)\right]$
c	Shear force at each section of the pile, *S* (N)	$S_1 = -H$	$S_2 = e^{\beta x}[(C_1 - C_2)\cos\beta x + (C_1 + C_2)\sin\beta x] + e^{-\beta x}[-(C_3 + C_4)\cos\beta x + (C_3 - C_4)\sin\beta x]$

5.7 BEARING SUPPORT SYSTEM

5.7.1 General

Revise Paragraph 2 of Clause (1) and Clause (4) as follows:

(1) Moreover, the bearing support systems shall also satisfy the design and detail requirements under each of the other limit states of the *DPWH Design Guidelines, Criteria & Standards (2015)* and the *AASHTO LRFD Bridge Design Specifications (2012 or later), Section 14 – Joints and Bearings*.

(4) The bearing support system shall be designed considering the applicable requirements of the structural details of the support specified in Article 5.7.4 of these Specifications and Section 14 of the *AASHTO LRFD Bridge Design Specifications* or when applicable, Section 19.1 of the *DPWH Design Guidelines, Criteria & Standards (2015)* in order to fully ensure functions of bearing support system. Bearings shall be consistent with the intended seismic response of the whole bridge.

Commentary C5.7.1

Revise Paragraph 1 of Commentary (1) as follows:

(1) These provisions for bearing support systems basically covers the requirements for extreme event limit state under earthquake effects. However, the support for other limit states under the *DPWH Design Guidelines, Criteria & Standards (2015)* and the *AASHTO LRFD Bridge Design Specifications* shall be complied with whenever applicable, especially the design movements under other load conditions including temperature changes and live load effects.

5.7.2 Design Seismic Force for Bearing Support System

Revise Clause (2) as follows:

(2) The design horizontal seismic force for Level 1 Earthquake Ground Motion shall correspond to the inertial force calculated by using the design horizontal seismic coefficients ~~(C_{sm})~~ (S_a) specified in Article ~~3.6~~ 3.4.1 of ~~these~~ the Interim Specifications multiplied by the dead load and factored live load reactions (with live load factor given in Article 1.5) when Type A bearing supports are used. No verification shall be needed for Type B bearing supports under Level 1 EGM. However, C_{sm} shall not be greater than 0.20, 0.25 and 0.30 for Ground Types I, II and III respectively.

Add Section 5.9 Expansion Joints and Commentary C5.9 as follows:

5.9 EXPANSION JOINTS

(1) An expansion joint shall be designed to ensure its functions in verification of its performance under Level I Earthquake Ground Motion. When an expansion joint is designed in accordance with Clause (2) or (3), this Clause (1) shall be regarded as satisfied.

(2) An expansion joint shall be designed to ensure that its expansion-contraction length will be larger than or equal to the design seismic expansion-contraction length that is calculated from Equation 5.9.1 for Level 1 Earthquake Ground Motion. However, if the design length specified in other limit states is larger than the calculated one, the design length during an earthquake shall not be less than the following value.

$L_{ER} = \delta_R + L_A$ (between a superstructure and an abutment) …………… (5.9-1)

$L_{ER} = c_B\,\delta_R + L_A$ (between two adjacent girders)

where,

L_{ER} : The design expansion-contraction length of an expansion joint during an earthquake (mm)

L_A : Allowance of expansion-contraction length (mm)

δ_R : Relative displacement between a superstructure and a substructure, occurring at the expansion joint location when subjected to Level I Earthquake Ground Motion (mm)

c_B : Modification factor for natural period difference on joint gap width. The values in Table 5.8-1 are based on the natural period difference ΔT between the two adjacent girders.

(3) An expansion joint shall be designed to ensure strength larger than or equal to sectional forces that occur in the main body of the expansion joint and its anchoring members under Level 1 Earthquake Ground Motion. In this case, the strength of the main body of the expansion joint and its anchoring members may be calculated by using the stress determined for an extra coefficient of 1.5.

Commentary C5.9

(1) An expansion joint shall be designed to maintain its functions under Level 1 Earthquake Ground Motion. For this purpose, its expansion-contraction length shall be larger than or equal to the relative displacement that occurs between two adjacent superstructures or between a superstructure and an abutment under Level 1 Earthquake Ground Motion, or horizontal resistance shall be ensured so that the main body of the expansion joint and its anchoring portion will not be damaged under Level 1 earthquake Ground Motion. For an expansion joint, verification of Seismic Performance Level 1 should be carried out in response to Level 1 Earthquake Ground Motion. It is acceptable that no verifications of Seismic Performances Level 2 and Level 3 in response to Level 2 Earthquake Ground Motion are performed. The reason for this is that even if the expansion joint has suffered some damages, the possibility to cause fatal damages to the bridge remains rather low, and the damages to the expansion joint can be covered by some temporally repair for traffic such as covering the road surface with steel plates. However, when it comes to be necessary to guarantee the traffic functions without such temporary repair work after a large-scale earthquake, it is recommended that adoption

of an expansion joint capable of dealing with a big displacement or an absorption system of displacement can be considered. Also, in order to make the recovery work of expansion joints easy, the adoption of an anchoring device structure that can be replaced simply to the expansion joint should be considered.

When carrying out a design for which the relative displacement between a superstructure and a substructure in the transverse direction to the bridge axis is taken into account, the design expansion-contraction length in the transverse direction to bridge axis during an earthquake should be also ensured. However, the expansion-contraction design lengths in the longitudinal and transverse directions to bridge axis should not be considered together. They can be considered separately. When it is not rational to make a joint gap for an expansion joint in the transverse direction to bridge axis, the expansion joint should be designed to ensure appropriate strength that allows the expansion joint to restrain the behavior in the transverse direction to the bridge axis under Level 1 Earthquake Ground Motion.

In addition, in a bridge that is elastically supported in the transverse direction to the bridge axis, large relative displacement may occur between adjacent superstructures in the transverse direction to the bridge axis for the reasons such as the differences in the types of superstructures and the span length. In such a case, the expansion joint may transmit the horizontal force. Consequently, under Level 2 Earthquake Ground Motion, large response displacement may occur in one superstructure, and thereby, displacement larger than assumed in design may be developed at the bearings of the other adjacent superstructure. To prevent this phenomenon from occurring, the type of expansion joint should be carefully selected. For example, the expansion joint should be designed not to allow horizontal force to be transmitted to the adjacent superstructure under Level 2 Earthquake Ground Motion. When the bridge is designed to contain an expansion joint that will allow horizontal force to be transmitted in the transverse direction to bridge axis under Level 2 Earthquake Ground Motion, the verification should be performed with appropriate consideration given to its influence on responses of adjacent superstructures.

(2) The calculation formula of the expansion-contraction design length during an earthquake is specified based on the minimum relative displacement between a superstructure and a substructure under Level 1 Earthquake Ground Motion., the value calculated Equation 5.9-1 is redefined as required value L_{ER} in order to distinguish this value from the expansion-contraction length that is actually ensured.

The following example describes how to calculate the maximum relative displacement δ_R occurring between a superstructure and a substructure at the location of the expansion joint.

1) In the case where supporting points are supported by elastomeric bearings

In the case where supporting points at the end of a superstructure with an expansion joint attached are supported by elastomeric bearings, the relative displacement occurs between the superstructure and the supporting substructure when Level 1 earthquake Ground Motion is exerted on a design vibration unit containing the supporting points at the end of the superstructure. The maximum value of this relative displacement is defined as δ_R. Generally δ_R may be regarded as the design horizontal displacement of the elastomeric bearing.

2) In the case where supporting points are supported by movable bearings

In the case where supporting points at the end of a superstructure with an expansion joint attached are supported by movable bearings, relative displacement occurs when Level 1 Earthquake Ground Motion is exerted on both a design vibration unit containing the superstructures and a design vibration unit containing the substructure. The maximum value of this relative displacement is defined as δ_R. However, since the deformation of abutment is generally small, the displacement of a design vibration unit consists of an abutment may be regarded as zero.

Relative displacement occurring between a superstructure and a substructure varies depending on various conditions. Therefore, when a dynamic verification method is applied to verify their seismic performance, relative displacement δ_R between the superstructure and the substructure may be determined on the basis of the analysis results.

Generally, the allowance of the expansion-contraction length for an expansion joint can be set around 15mm towards the respective directions of expansion-contraction. As to the allowance of the expansion-contraction length for the expansion joint which has been considered for normal loads, 10mm shall be taken as the standard. However, considering the margins of error in the calculation of the expansion-contraction length during an earthquake as well as the margin of error during construction, it is recommended that 15mm is taken as the standard allowance of the expansion-contraction length.

In order to avoid the interference in expansion-contraction caused by the application of the normal loads, if the design expansion-contraction length during earthquake which is obtained by Equation 5.9-1 turns out to be smaller than that at normal time, the design expansion-contraction length shall be based on the expansion-contraction length under normal conditions. The expansion-contraction length under other limit states can be calculated by following the provisions of the *DPWH Design Guidelines, Criteria & Standards (2015)* or the *AASHTO LRFD Bridge Design Specifications.*

(3) This clause specifies that in order to ensure functions of an expansion joint under Level I Earthquake Ground Motion, the expansion joint should have strength larger than or equal to the sectional forces that occur in the main body of the expansion joint and its anchoring members under Level I Earthquake Ground Motion. Accordingly, the design seismic force that occurs in the main body of the expansion joint and anchoring members under Level I Earthquake Ground Motion should be appropriately determined with consideration given to conditions that are disadvantageous from the viewpoint of the design of the expansion joint. The strength of the main body of the expansion joint and its anchoring members should be determined for the limit state of allowing the expansion joint to stay in the elastic range so as to prevent the expansion joint from being damaged.

SECTION 6: EFFECTS OF SEISMICALLY UNSTABLE GROUND

6.1 GEOTECHNICAL PARAMETERS OF EXTREMELY SOFT LAYER AND SANDY LAYER PRONE TO LIQUEFACTION

6.2.3 Assessment of Soil Liquefaction

Revise the definition of k_{hgL} in Clause (2) to include Level 1 EGM as follows:

(2) Assessment of Liquefaction

k_{hgL} : Design horizontal seismic coefficient at the ground surface for Level 1 and Level 2 EGM.

Commentary C6.2.3

Revise Commentary (2) as follows:

(2) Liquefaction assessment shall be performed for Level 1 and Level 2 Earthquake Ground Motion. Since the cyclic triaxial shear stress ratio R_L changes greatly according to the repeated property of seismic ground motions, the ratio shall be modified by Equation 6.2.3-8.

6.2.4 Reduction of Geotechnical Parameters

Revise Table 6.2.4-1 to represent the factors for Level 1 and Level 2 EGM as follows:

Table 6.2.4-1 Reduction Factor D_E for Geotechnical Parameters of Level 1 and Level 2 EGM

Range of F_L	Depth from Present Ground Surface x (m)	Dynamic Shear Strength Ratio, R	
		$R \leq 0.3$	$0.3 < R$
		~~Verification for Level 2 Earthquake Ground Motion~~	~~Verification for Level 2 Earthquake Ground Motion~~
$F_L \leq 1/3$	$0 \leq x \leq 10$	0	1/6
	$10 < x \leq 20$	1/3	1/3
$1/3 < F_L \leq 2/3$	$0 \leq x \leq 10$	1/3	2/3
	$10 < x \leq 20$	2/3	2/3
$2/3 < F_L \leq 1$	$0 \leq x \leq 10$	2/3	1
	$10 < x \leq 20$	1	1

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SECTION 7: REQUIREMENTS FOR UNSEATING PREVENTION SYSTEM

7.1 SEAT LENGTH

Clauses (3) and (4) for skew and curved bridges are deleted and combined to revised Clause (3) as follows:

~~(3) For a skew bridge with superstructure shape satisfying Equation 7.5-1, the seat length shall satisfy the provisions in (1) and be calculated by Equation 7.2-4. For an asymmetric skew bridge in which the two front lines of the bearing supports at both ends of the superstructure are not parallel,~~ $\cancel{S_{E\theta}}$ ~~shall be calculated with the use of a smaller skew angle.~~

$$\cancel{S_{E\theta} \geq (L_\theta/2)(\sin\theta - \sin(\theta - \alpha_E))} \qquad \cancel{(7.2\text{-}4)}$$

~~where:~~

$\cancel{S_{E\theta}}$: ~~Seat length for a skew bridge, (m).~~

$\cancel{L_\theta}$: ~~Length of a continuous superstructure, (m).~~

$\cancel{\theta}$: ~~Skew angle, (degree).~~

$\cancel{\alpha_E}$: ~~Marginal unseating rotation angle, (degree).~~ $\cancel{\alpha_E}$ ~~can generally be taken as 5 degrees.~~

~~(4) For a curved bridge with superstructure shape satisfying Equation 7.5-2, the seat length shall satisfy the provisions in (1) and be calculated by Equation 7.2-5.~~

$$\cancel{S_{E\phi} \geq \delta_E \frac{\sin\phi}{\cos(\phi/2)} + 0.3} \qquad \cancel{(7.2\text{-}5)}$$

$$\cancel{\delta_E \geq 0.005\phi + 0.70} \qquad \cancel{(7.2\text{-}6)}$$

~~where:~~

$\cancel{S_{E\phi}}$: ~~Seat length for a curved bridge, (m).~~

$\cancel{\delta_E}$: ~~Displacement of the superstructure toward the outside direction of the curve, (m).~~

$\cancel{\phi}$: ~~Fan-shaped angle by the two edges of a continuous girder of a curved bridge, (degrees).~~

(3) For one or two-span bridges which may collapse when the superstructure moves in the transverse direction to the bridge axis due to the nature of the superstructures that have the possibility to rotate due to its structural condition or geometric configuration and not constrained by the adjacent girder or the abutment, its girder seat length shall satisfy Equation 7.2-1 but shall larger than or equal to the value calculated by Equation 7.2-4.

For an asymmetric skew bridge in which the two front lines of the bearing supports at both ends of the superstructure are not parallel, $S_{E\theta R}$ shall be calculated with the use of a smaller skew angle.

$$S_{E\theta R} = 2L_\theta \sin(\alpha_E/2)\cos(\alpha_E/2 - \theta) \qquad (7.2\text{-}4)$$

where:

$S_{E\theta R}$: Required girder seat length of a bridge defined in Clause (3) above, (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 2.5 degrees.

Commentary C7.2

Commentary (3) and (4) are deleted and replaced by Clause (3) and Figures C7.2-4 and C7.2-5 replaced by new Figure C7.2-4 as follows:

~~(3) The seat length of a skew bridge shall be determined by considering the rotation of the superstructure, since unseating of the bridge may happen as a result of the structural behavior illustrated in Figure C7.2-4. For skew bridges with skew angle, length and width of a continuous superstructure satisfying Equation 7.5-1, including both simple girder bridges and continuous bridges, the bridge unseating may be caused by the rotation. In this case, the seat length~~ ~~$S_{E\theta}$~~ ~~for rotation shall be determined based on limit of unseating rotation angle~~ ~~α_E~~~~, and the larger one obtained from Articles (1) or (2) of this commentary shall be taken as the seat length.~~

~~In Equation 7.2-4, only the skew bridge rotating around its center of gravity, which is limited by the unseating rotation angle, is included in which the center of the superstructure end comes out of the seat length. The limit of unseating rotation angle can generally be taken as 5 degrees, according to examples of damage during the 1995 Hyogo-Ken Nambu Earthquake (Japan), and the results of dynamic analysis for various bridge types using earthquake ground motions obtained from the event.~~

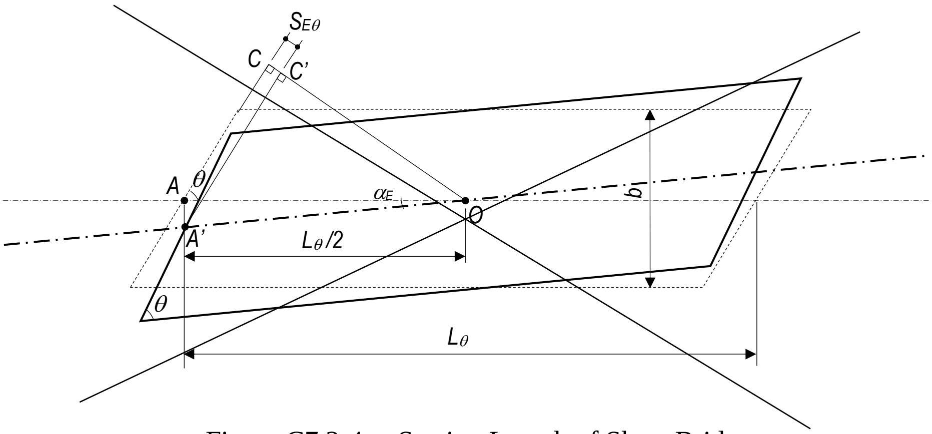
~~The limit of unseating rotation angle~~ ~~α_E~~ ~~shall be determined when the skew angle shall be made unavoidably narrower on a multi-span continuous bridge. For a multi-span continuous bridge with longer length of a continuous superstructure~~ ~~L_θ~~~~, the value of the seat length~~ ~~$S_{E\theta}$~~ ~~obtained from Equation 7.2-4 may be quite large. In this case, the seat length may lead to a considerably irrational structure for the entire bridge. As a result, measures against such disadvantage should be taken, such as reexamining an appropriate limit of unseating rotation angle as described above, or making the structure limiting excessive displacement in the transverse direction to the bridge axis having the same strength as that of unseating prevention structure specified in Article 7.3 so that the rotational displacement of the superstructure could surely be restrained. In addition, the seat length in this case should generally be greater than 1.5 times the value obtained from Articles (1) or (2) of this commentary corresponding to different structural types and scales of the bridges.~~

~~The seat length shall be determined using the smaller skew angle, assuming the bridge rotating around the center line of the bridge axis, when the bearing lines on the two sides of the superstructure are not parallel, and when the skew angles at both ends are different.~~

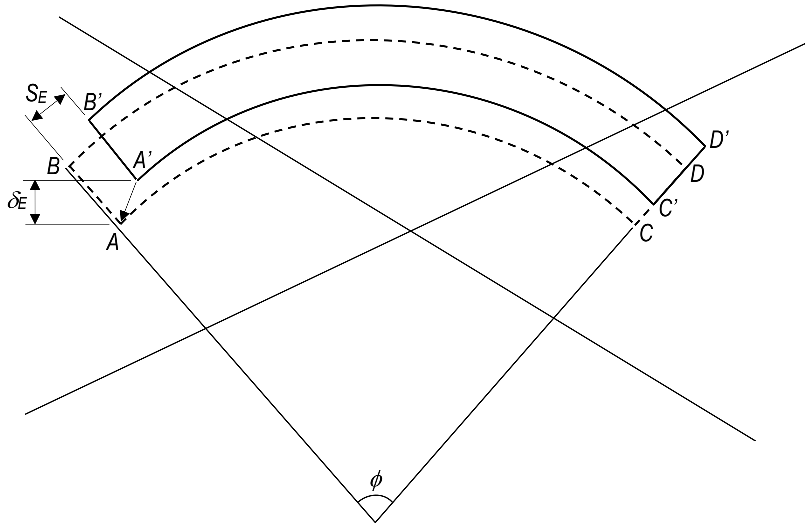
~~(4) For reason of structural characteristics shown in Figure C7.2-5, curved bridges are more prone to unseating due to rotation of the superstructure, or moving toward the outside of the curve. Consequently, the seat length shall be determined taking these effects into consideration. The superstructure may start rotating without being restricted by the abutment or the adjoining superstructures when the intersection angle, the length and the width of the series of superstructures satisfies Equation 7.5-2. In addition, the superstructure might move toward the outside of the curve line due to complicated vibrations occurring during an earthquake, accompanied by a danger of the superstructure falling off the top of the substructure. Since the influence of the intersection angle is dominant during this type of movement, the limit of unseating rotation angle shall be determined from the intersection angle, and the seat length shall be calculated using Equation 7.2-5. Similarly, the seat length in the skew bridges obtained by this means shall be compared with the values obtained from Articles (1) or (2) of this commentary and the~~

~~larger one shall be taken. When determining the seat length for continuous curved bridges, the intersection angle between the two girders may be large enough to consider careful attention to prevent superstructure rotation or movement toward the outside of the curve line of the superstructure.~~

~~In case that the seating length $S_{E\phi}$, obtained from Equation 7.2-5 is large so that a considerably irrational structure in the entire bridge is formed, measures against such disadvantage should be taken so that the rotation or movement toward the outside of the curve line of the curved bridges can be properly restrained. Such measures include providing a device or structure limiting excessive displacement in the transverse direction to the bridge axis having the same strength as that of an unseating prevention device specified in Article 7.3. In addition, the seat length in this case should generally be greater than 1.5 times the value obtained from Articles (1) or (2) of this commentary corresponding to different structural types and scales of the bridges.~~



~~Figure C7.2-4 Seating Length of Skew Bridge~~



~~Figure C7.2-5 Seating Length Corresponding to the Movement of Curved Bridge~~

(3) For one or two span bridges that has the possibility to rotate due to the structural and geometric conditions of the superstructures and with the superstructure not being constrained by the adjacent girders or the abutment, the superstructure may fall-down or collapse after the bearing support is damaged or destroyed, as shown in Figure C7.2-4. Therefore, this Clause specifies that the girder seat length should be determined with consideration given to the influence of rotation. According to the provision, in this process, required girder seat length $S_{E\theta R}$ corresponding to the rotation shall be

calculated from the critical unseating rotation angle α_E, and the required girder seat length shall be set to the larger value of $S_{E\theta R}$ and the girder seat length obtained in accordance with Clause (1).

Equation 7.2-4 yields $S_{E\theta R}$ corresponding to the situation in which a bridge rotates around a girder by the critical unseating rotation angle to cause the other end of the superstructure to deviate from the girder seat length. Previous versions specified the condition in which a skew bridge would rotate around the center of gravity of its superstructure. In this revision, Equation 7.2-4 is adopted on the basis of the characteristics of the rotation behavior of superstructures during an earthquake in order to apply it to superstructures that have a skew angle at their girder ends, regardless of their planar shapes. For a curved bridge, skew angle θ' that is used for evaluating the condition for allowing curved bridges to rotate as shown in Figure C7.2-4 is used as the skew angle in Equation 7.2-4. In this process, the required girder length needs to be determined with consideration given to the degree of rotation of the bridge during an earthquake. However, technical information and knowledge of the behavior of bridges after failure or destruction of the bearing support have not been sufficiently accumulated. Therefore, critical unseating rotation angle α_E in Equation 7.2-4 has been refined so that the girder seat length will approach the previous girder seat length that was determined on the basis of both the damage cases of the 1995 Hyogo-ken Nanbu Earthquake and the dynamic analysis results of various types of bridges.

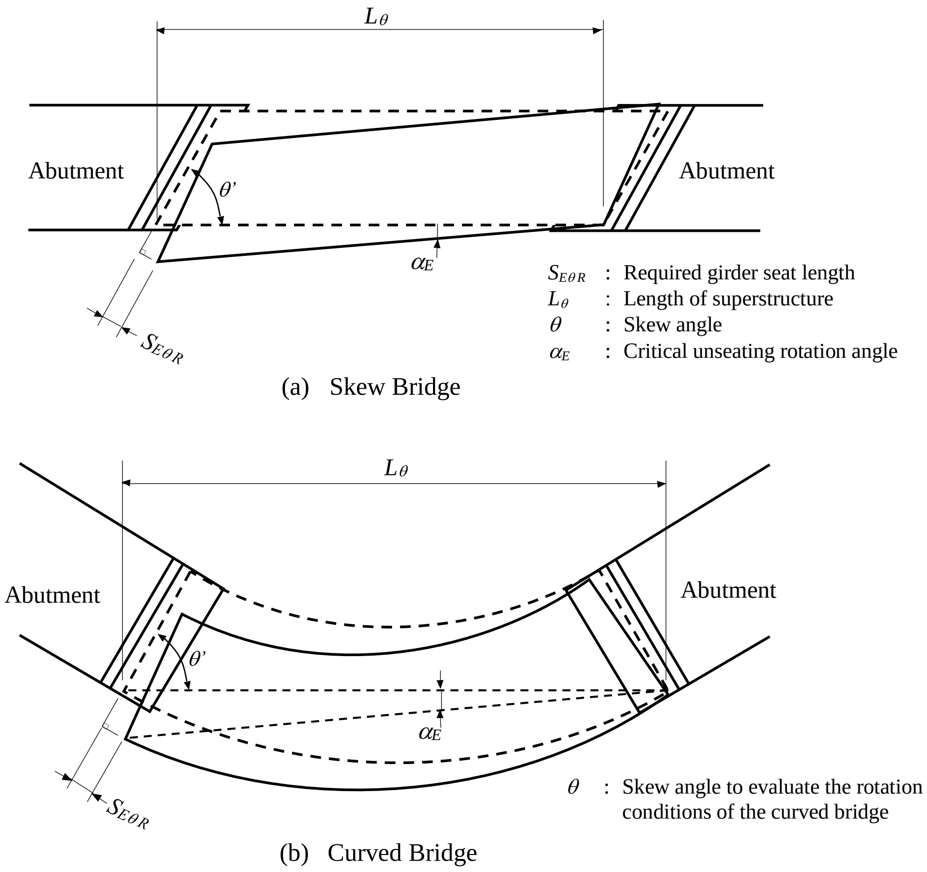


Figure C7.2-4 Required Girder Seat Length for Bridges with Possibility to Rotate

7.5 STRUCTURE OR DEVICE TO LIMIT EXCESSIVE DISPLACEMENT

Equation 7.5-2 in Clause (1) 2) replaced and definition θ' inserted as follows:

(1) 2) Curve bridges satisfying Equation 7.5-2.

$$\cancel{\frac{115}{\phi}\frac{(1-\cos\phi)}{(1+\cos\phi)} > b/L}$$

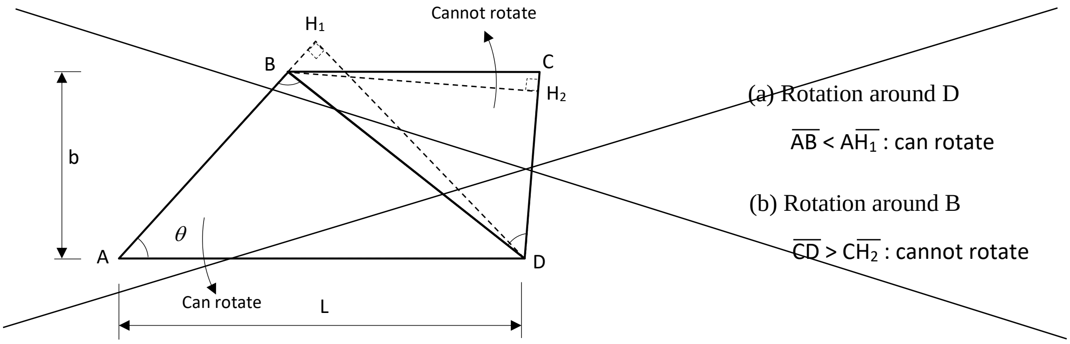
$\underline{\cos\theta' > b/L}$ .... (7.5-2)

where:

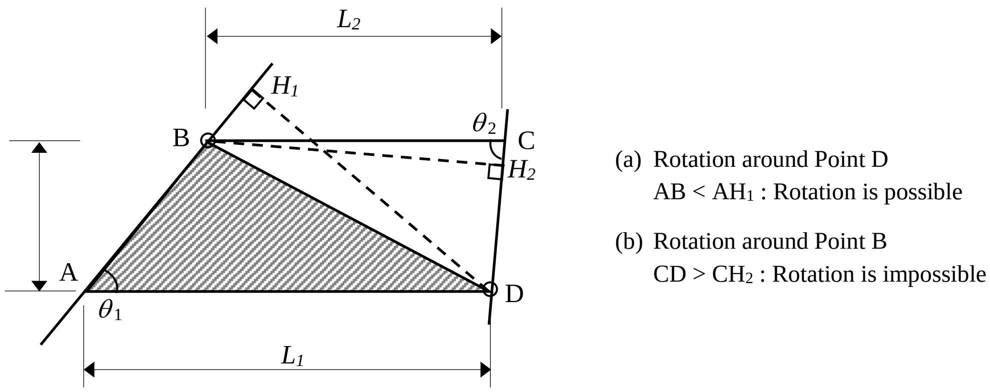
$\underline{\theta'}$: <u>Skew angle used for evaluating the condition for rotation of a curved bridge , (degree).</u>

Commentary C7.5

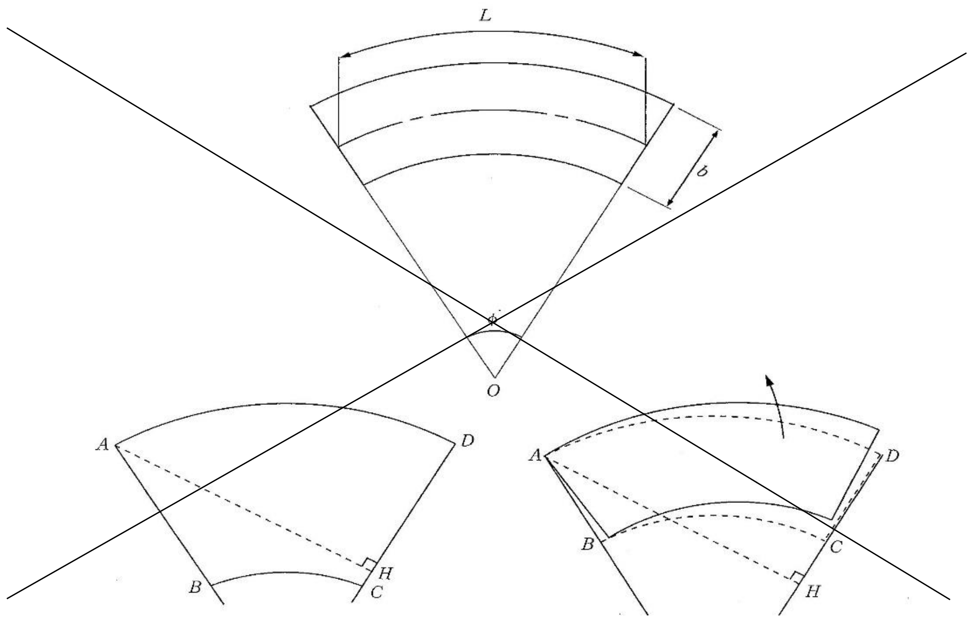
Replace Figure C7.5-2, C7.5-3 and C7.5-5 with revised Figures C7.5-2, C7.5-3 and C7.5-5 as follows:



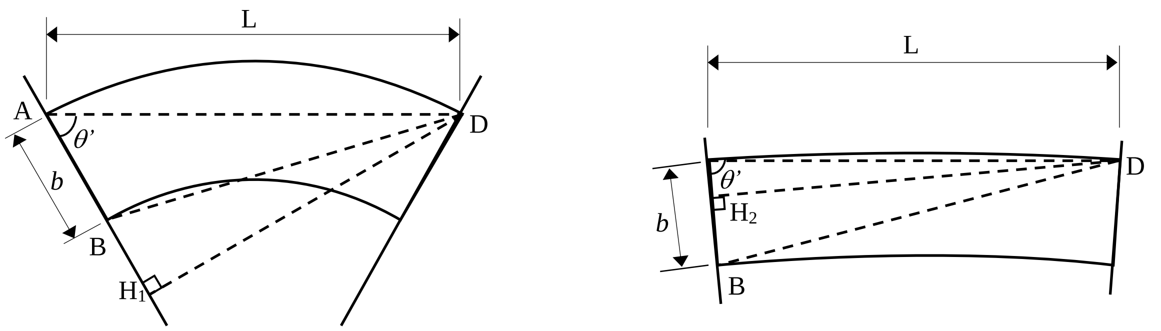
~~Figure C7.5-2 Conditions in which a Skew Bridge with Unparallel Bearing Lines on Both Edges of the Superstructure can Rotate.~~



<u>Figure C7.5-2 Condition for Allowing Skew Bridge to Rotate Without Being Constrained by Adjacent Girders or Abutments</u>

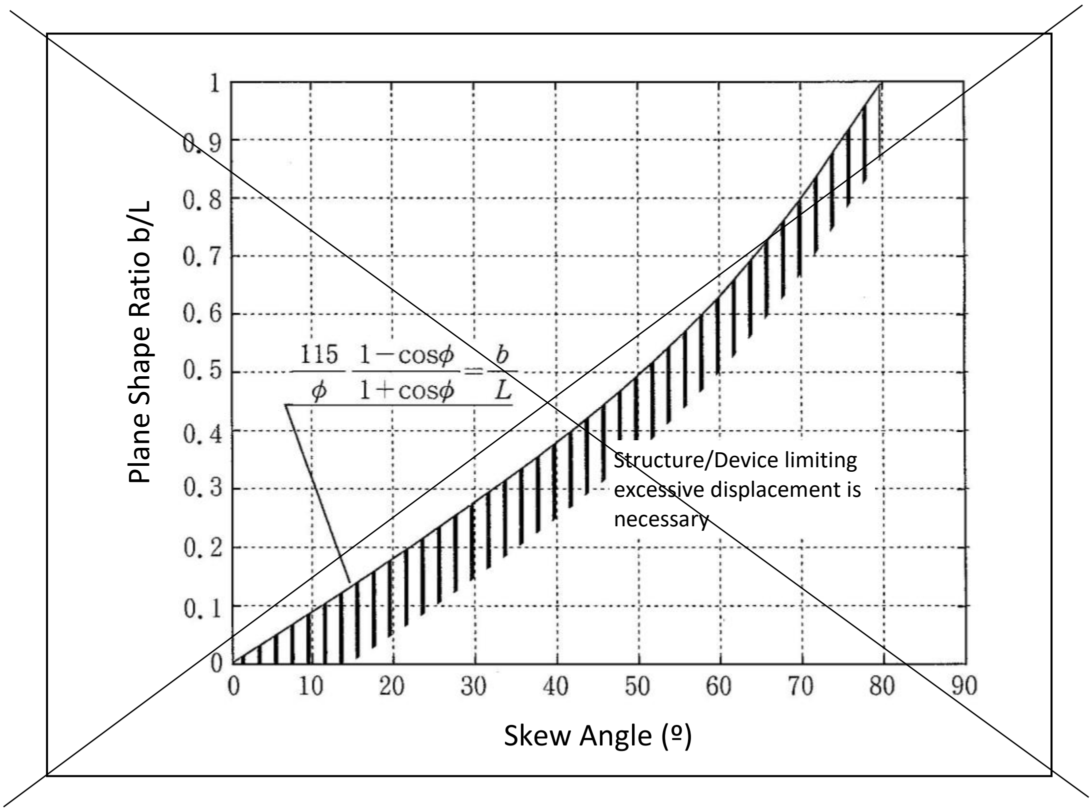


~~Figure C7.5-3 Conditions in which a Curved Bridge can Rotate Without Being Affected by Adjoining Girders or Abutment~~



(a) When rotation is possible: AB < AH_1 (b) When rotation is impossible: AB > AH_2

Figure C7.5-3 Condition for Allowing Curved Bridge to Rotate Without Being Constrained by Adjacent Girders or Abutments



~~Figure C7.5-5 Conditions in which a Curved Bridge Requires a Structure Limiting Excessive Displacement in the Transverse Direction to the Bridge Axis~~

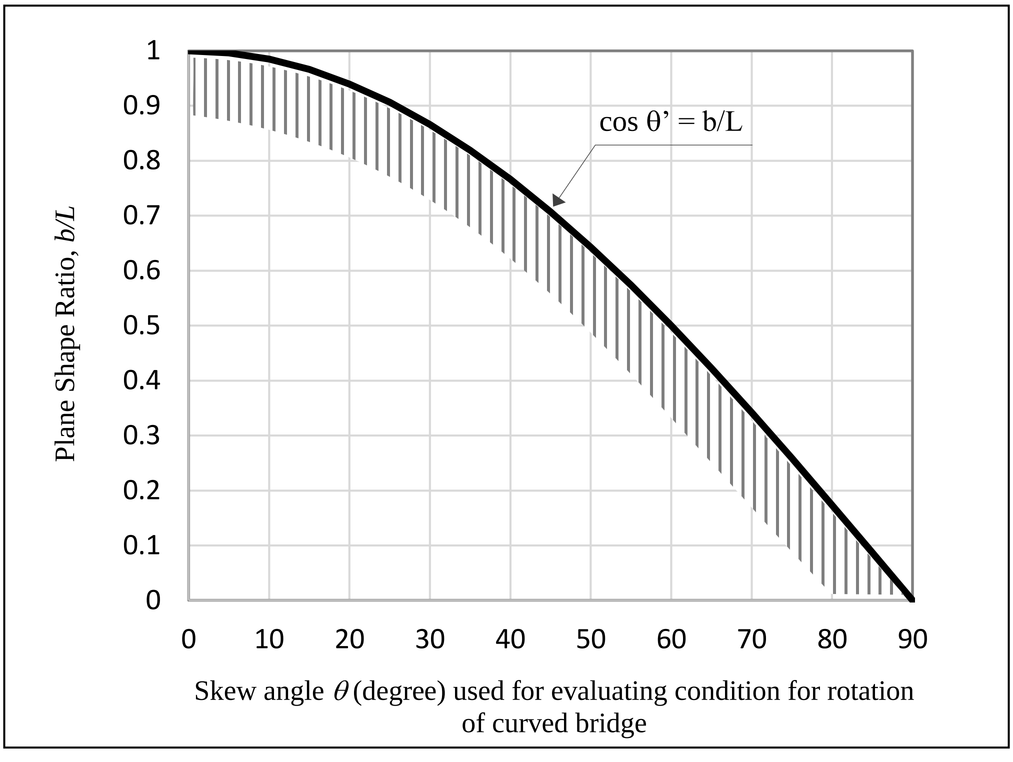


Figure C7.5-5 Condition for Installation of Lateral Displacement Confining Structure in Curved Bridge in the Transverse Direction to the Bridge Axis

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SECTION 8: REQUIREMENTS FOR SEISMICALLY ISOLATED BRIDGES

8.1 GENERAL

Insert Clause (7) as follows:

(7) The *Road Bridge Seismic Isolation Design Guideline, DPWH 2018 (1st Ed.)* supplemental specification is prepared to cover the seismic isolation design of new bridges and to provide the design procedures for laminated rubber bearing, laminated rubber bearing with lead plug and high damping rubber bearing.

Commentary C8.1

Insert Comment (7) as follows:

(7) BSDS Section 8 covers the general requirements and procedures for the design of seismically isolated bridges. However, the need to provide a more detailed guidelines on seismic isolation design lead to the development of the *Road Bridge Seismic Isolation Design Guideline, DPWH 2018 (1st Ed.)* focusing on the seismic performance verification of seismically isolated bridges, the method for dynamic verification and the verification of bearing supports. Moreover, the design verification for three types of bearing systems including laminated rubber bearing, laminated rubber bearing with lead plug and high damping laminated rubber bearing are introduced. Design examples for seismically isolated bridge using these bearings are also introduced.

8.2 PERFORMANCE VERIFICATION OF SEISMICALLY ISOLATED BRIDGES

Revise Section 8.2 as follows:

(1) The verification for seismic performance of a seismically-isolated bridge shall be done by dynamic analysis methods depending on the bridge properties, structural configurations and complexity of seismic behavior or by static analysis method as approved by DPWH. Bridges without complicated seismic behavior may be analyzed by static method while bridges with complicated seismic behavior shall be analyzed by dynamic method. When calculating the seismic response of a seismically-isolated bridge, the isolation bearing can be modeled in accordance with the provisions given in Article 8.3 of this Section. The requirements in Clause (2) shall be followed to verify seismically isolated bridges ensuring Seismic Performance Level 1 (SPL1), and the requirements in Clause (3) shall be followed to verify seismically isolated bridges ensuring Seismic Performance Level 2 or 3 (SPL2 or SPL3).

(2) The requirements in Section 4 of this Specifications shall be followed to verify seismically isolated bridges ensuring Seismic Performance Level 1 (SPL1) by dynamic verification method. When verifying seismically isolated bridges ensuring Seismic Performance Level 1 (SPL1) by static verification method, appropriate models shall be used based on the dynamic behaviors of seismic isolation bearings.

(3) To ensure the Seismic Performance Level 2 or 3, ~~The~~ the allowable ductility ~~ratio~~ factor of reinforced concrete columns in a seismically-isolated bridge shall be obtained from Equation 8.2-1.

$$\cancel{\mu_m = 1 + \frac{\delta_u - \delta_y}{\alpha_m \delta_y}} \quad \mu_m = \frac{\delta_{ls2}}{\alpha_m \delta_y} \quad \text{.......} \quad (8.2\text{-}1)$$

Note μ_m] 1.0

where:

μ_m : Allowable ductility ~~ratio~~ factor of reinforced concrete columns in seismically-isolated bridge.

α_m : Safety factor used for the calculation of the reinforced concrete columns. α_m shall be calculated by Equation 8.2-2.

$$\cancel{\alpha_m = 2\alpha} \quad \alpha_m = 2\alpha_2 \quad \text{.......} \quad (8.2\text{-}2)$$

~~α~~ α_2 : Safety factor used for calculation of the allowable ductility ratio of the reinforced concrete columns for Seismic Performance Level 2 or 3, and is specified ~~in Table 8.2-1~~ as 1.2 in general.

~~Table 8.2.1 Safety Factor α for Calculating Ductility Capacity of a Reinforced Concrete Column Resulting in Flexural Failure~~

~~Seismic Performance to be Verified~~	~~Safety Factor α for Calculating Ductility Capacity~~	
	~~Plate Boundary Type Earthquake Ground Motion (Type I)~~	~~Inland Direct Strike Type Earthquake Ground Motion (Type II)~~
~~Seismic Performance Level 2 (SPL-2)~~	~~3.0~~	~~1.5~~
~~Seismic Performance Level 3 (SPL-3)~~	~~2.4~~	~~1.2~~

~~δ_y, δ_u~~ δ_y, δ_{ls2} : Yield displacement and ultimate displacement of the reinforced concrete columns for Seismic Performance Level 2 or, respectively.

(4) Isolation bearings shall b verified in accordance with provisions in Article 5.7. In addition, only isolation bearings having the fundamental functions specified in Article 8.4 of this Section shall be basically selected.

(5) Design of superstructure end of a seismically-isolated bridge shall be based on Article 5.8 of these Specifications.

Commentary 8.2

Rewrite Commentary (1) as follows:

(1) Although verification of the seismic performance of a seismically-isolated bridge shall be done based on Article 3.3, proper considerations of the bridge properties shall be included in the verification. For a seismically-isolated bridge subjected to Level 2 Earthquake Ground Motion, the nonlinearity of the isolation bearing and the secondary plasticity of the pier shall be taken into account. A seismically-isolated bridge shall be classified as a bridge having complex seismic behaviors during an earthquake and verification of such bridge shall be carried out by dynamic analysis method. During the verification process by dynamic analysis method, the dynamic analysis procedures shall be utilized while the items listed below shall be confirmed at the same time.

1) The displacement shall be absorbed mainly by the isolation bearings and no excessive displacement shall be concentrated in the substructure.

2) The damping performance of the bridge shall be improved by the isolation bearings.

3) ~~Moreover~~, For a typical seismically-isolated bridge, once the first mode which basically consist of the displacement of the isolation bearing becomes dominant, the vibration characteristics of all the piers will turn out to be roughly the same in most cases. Therefore, during the process of modeling, unnecessarily complicated models should be avoided. Each pier can be separated, when necessary, in order to create a model that is capable of appropriately reflecting the vibration characteristics of the bridge.

4) When verifying a seismically isolated bridge ensuring Seismic Performance Level 1 (SPL1) by the dynamic verification method in consideration of the nonlinear hysteretic characteristics of seismic isolation bearings, the same model used to verify a seismically isolated bridge ensuring Seismic Performance Level 2 or 3 (SPL2 or SPL3) may be used. However, when modeling a seismic isolation bearing as an equivalent linear member, it is necessary to appropriately define equivalent stiffness and an equivalent damping ratio based on displacement generated in the area with seismic isolation bearings designed against Level 1 Earthquake Ground Motion. When verifying by the static verification method, seismic isolation bearings should be modeled generally as a linear member with equivalent stuffiness.

5) The energy absorption of seismic isolation bearings and the limited plasticity of piers are taken into account for a seismically isolated bridge against Level 2 Earthquake Ground Motion. Therefore, because the bridge is thought to exhibit complex behaviors during an earthquake, as mentioned in Section 4.3 of this Specifications, the dynamic method shall be used for verification. It is necessary to set the model of a seismic isolation bearing used for dynamic analysis so that the relationship between the horizontal load and horizontal displacement of the seismic isolation bearing can be evaluated appropriately in light of various conditions and dynamic characteristics of seismic isolation bearings to be used. Verification is necessary to ensure that mainly the seismic isolation bearings absorb deformation and energy, and that substructures remain plastic to a limited degree, as expected of a seismically isolated bridge. Reference is given to the *DPWH Guide Specifications for Seismic Isolation Design*.

   As a specific verification method, it is specified to conduct verification with the use of the allowable ductility factor of a reinforced concrete pier, derived by halving the allowable ductility factor of a reinforced concrete pier of a bridge that has the function of distributing a horizontal load during an earthquake. This is intended to control responses generated in the reinforced concrete pier column to limited plastic deformation, and is also intended to minimize damage and make certain that the natural period of seismic isolation bearings, not piers, will become longer, and energy will be absorbed from seismic isolation bearings, not from piers. In this revision, an equation for calculating the allowable ductility factor of a reinforced concrete pier has been reviewed, and the allowable ductility factor may become 1.0 or less based on Equation 8.2-1. It is also specified that 1.0 can be used in this case. This is because it is permissible for responses in piers to remain within the elastic range as long as damage to the piers can be reduced. There are some who believe a limited degree of ductility should be allowed to reduce damage to piers. However, because there are insufficient findings about the degree of ductility, the ductility ratio has been specified as described above.

It is also specified that the allowable ductility factor of a reinforced concrete pier of a seismically isolated bridge shall be obtained for the allowable ductility factor for Seismic Performance Level 2 in both cases to verify Seismic Performance Level 2 or 3. This is because the permissible deformation of a reinforced concrete pier of a seismically isolated bridge is designed so that the plastic deformation of the pier can be controlled to a limited extent and so that seismic isolation bearings can definitely absorb energy. In this respect, the permissible plastic deformation of a reinforced concrete pier is the same for both Seismic Performance Level 3 and 2.

Here, the allowable ductility factor of a steel pier is not provided. The reason for this is that since a bridge with steel piers generally has a long natural period and in many cases it is not reasonable to adopt seismic isolation design for such a bridge. In cases where seismic isolation design is introduced for a bridge with steel piers, it is necessary to set the allowable ductility factor individually after making certain that seismic isolation bearings can definitely absorb energy.

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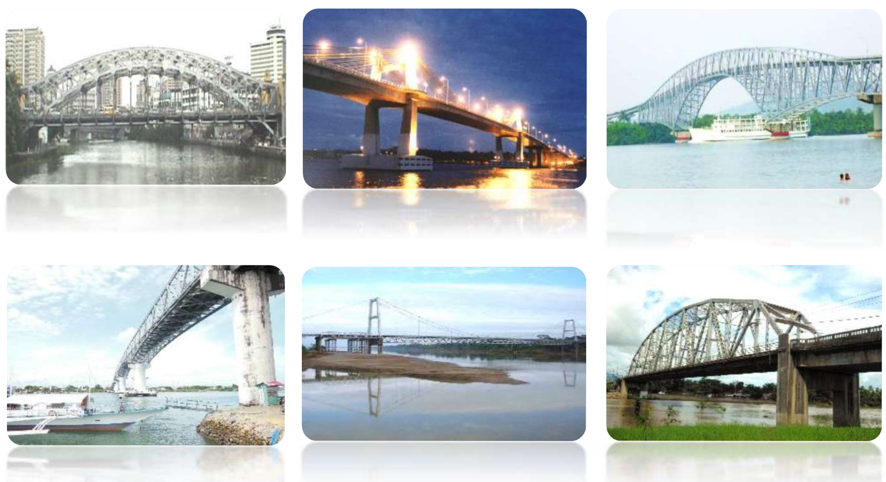

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LRFD Bridge Seismic Design Specifications
2018 Interim Revisions