REVISED NEW RECOMMENDED SEISMIC DESIGN GUIDE SPECIFICATIONS OF THE U.S. HIGHWAY BRIDGES

W. Phillip Yen[1] and Rich Pratt[2]

Abstract

This paper briefly discussed the major changes of the revised recommendations for AASHTO's New Bridge Seismic Design Guide Specifications. The revised recommendations was completed by a National Corporation Highway Research Program by the project 20-7/ (193) in May 2006, and will be considered for adoption in 2007.

Introduction

The performance of US highway bridges in recent large earthquakes has shown good design details have saved many bridges from collapsing due to unseating of superstructure or shear failure of columns. Seismic design methods have evolved over the past 30 years and have produced details that directly affect bridge performance under earthquake and other natural hazard loading. Design methods are steadily improved based on experience with destructive earthquakes and advanced seismic research. The current seismic design specification, adopted as a standard in 1992 by AASHTO, was primarily developed by US highway agencies, including FHWA and CALTRANS. Realistic seismic provisions first entered this code after the 1971 San Fernando earthquake. The fundamental design objective of the current seismic specifications is to prevent collapse in large earthquakes. In small to moderate earthquakes, the intent of the code is to resist these loads within the elastic range without significant damage to structural components. In large earthquakes, no span or part of a span should collapse. However, the AASHTO specs consider some damage acceptable in these circumstances, provided it is limited to flexural hinging in pier columns and that it occurs above ground in regions that are visible and accessible for inspection and repair.

The current earthquake design is a single level event with a 475-year return period. Design forces are calculated from an elastic analysis of the bridge using response spectra approximating the design quake. As the result of an effort by the National Cooperative Highway Research Program and FHWA, a recommended new seismic design criteria was completed in March, 2001. This recommended criterion contains significant changes in the design approach and criteria to reflect lessons learned from recent earthquakes and research studies. A dual-level design method was introduced in the recommended design

[1] Research Structural Engineer & Seismic Research Program Manager, Office of Infrastructure, R&D, Federal Highway Administration, McLean, VA
[2] Chief Bridge Engineer & AASHTO Bridge T3 Committee Chair, Alaska Department of Transportation, Juneau, AK

criteria. Bridge design objectives are categorized in two levels of seismic performance. They are “Life Safety” and “Operational”. However, this first recommendation was not adopted by AASHTO’s Subcommittee of Bridge and Structure in 2002. This resulted the Task Committee 3, Bridge Seismic Design, of this Subcommittee (called T3) requested a new study to revise this new design recommendations based on the review comments. This revised recommendation was completed in May 2006. This paper is to briefly introduce the main changes of this revised recommendation from NCHRP 12-49.

<u>Design Performance Criteria</u>

Under this revised recommendations, the seismic performance criteria is recommended for the life safety objective and considering a one level design rather than two-level design, Functional and Life Safety, in the recommendations of NCHRP 12-49 And the This single level design criteria is based on a 5% probability of exceedance in 50 years. Higher level performance such as the operational objectives may be used with the authorizations of the bridge owners. The following is the comparison with the current, NCHRP and revised recommendations of the performance criteria.

The performance objective of Life Safety for the Design Event is similar to NCHRP 12-49. It means that the bridge suffers significant damage and significant disruption to service. It should not collapse but partial or complete replacement may be required. And the significant Disruption to Service Level includes limited access (reduced lanes, light emergency traffic) on the bridge. Significant Damage Level includes permanent offsets and damage consisting of cracking, reinforcement yield, major spalling of concrete and 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. These conditions may require closure to repair the damage. Partial or complete replacement of columns
may be required in some cases. For sites with lateral flow due to liquefaction, significant inelastic deformation is permitted in the piles. Partial or complete replacement of the columns and piles may be necessary if significant lateral flow occurs. If replacement of columns or other components is to be avoided, the design approaches producing minimal or moderate damage such as seismic isolation or the control and repairability design concept should be assessed.

<u>Seismic Design Category</u>

The revised recommendations classify bridge seismic design into four different categories, i.e. Seismic Design Category (SDC) : A, B, C & D. The name of SDC is to distinguish bridge seismic design and retrofitting procedures. This SDC is similar to what is used in the current LRFD design provisions – Seismic Performance Category (SPC). However, the revised recommendations define these four SDCs as in the Table 1.

Table 1: Partitions for Seismic Design Categories A, B, C and D

Value of SD1	SDC
SD1 < 0.15g	**A**
0.15g ≤ *SD1*< 0.30g	**B**
0.30g ≤ *SD1*< 0.50g	**C**
0.50g ≤ *SD1*	**D**

Where *SD1* is recognized as the Design Spectral Acceleration based on the one-second period.

Earthquake Resisting Systems (ERS) and Earthquake Resistance Elements (ERE)

There are additional requirements for bridges located in the SDC C or D categories. These bridges and their foundations shall have a clearly identifiable Earthquake Resisting System (ERS) selected to achieve the Life Safety Criteria as defined in above. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen mechanism of seismic resistance and other structural requirements. For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance by the owner, the ERS and earthquake resisting elements (ERE) are categorized as follows: a)Permissible, b) Permissible with Owner's Attention and c) Not Recommended for New Bridges.

These terms apply to both systems and elements. For a system to be in the permissible category, its primary ERE must all be in the permissible category. If any ERE is not permissible, then the entire system is not permissible. Many EREs classified with Permissible, Permissible with Owner's Attention and Not Recommended for New Bridges were illustrated in the revised recommendations. In general, the permissible systems and elements have the two characteristics: 1) All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair; 2) Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g. cap beam and superstructure hinging).

Design Ground Motion

Either a general procedure or a site-specific procedure is allowed to generate design spectra for the design forces. However, for bridge located in the site class F soil, or for those important bridges designed for the higher performance objective, or bridge sites within 10km of a known active fault where a response is expected to be significant, a site-specific design procedure should be used.

Design Spectra

Design spectra based on general procedures are constructed differently from procedures in the current design codes. Figure 1 illustrates the so-called "Two Points Method" for constructing design spectra.

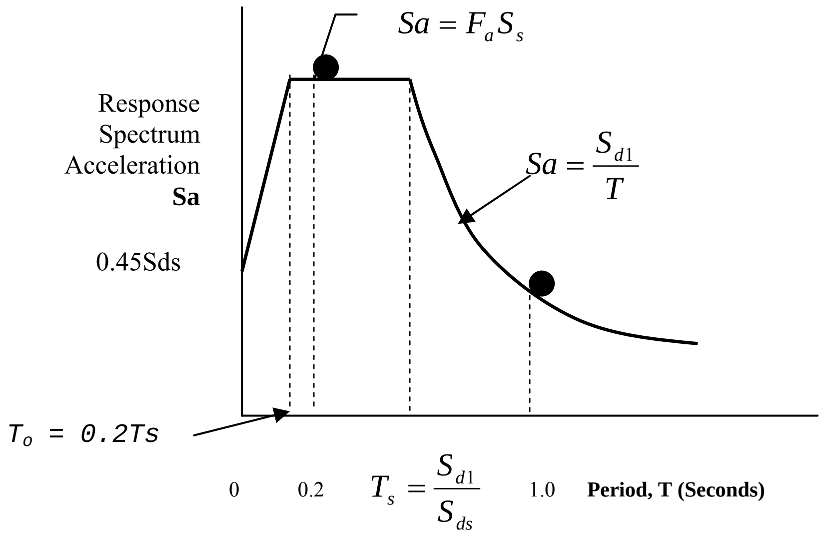

Figure 1. Seismic Design Spectra by Two Points Method

The design response spectrum curve for short periods, which is less than or equal to T_o, the design response spectral acceleration, *Sa,* shall be defined by the following equation:

$$Sa = 0.60\frac{Sds}{To}T + 0.40Sds$$

For periods between T_o and T_s, including T_o and T_s, the design response spectral acceleration, Sa is defined as $Sa = Sds$, and

For periods are greater than T_s, then $Sa = \frac{S_{d1}}{T}$

Where: S_{d1} is defined as $S_{d1} = F_v S_1$ (S_1 can be obtained from the USGS ground motion maps, and F_v is the site coefficient)

Soil Site Class Definitions

For classifying soil site, the average shear wave velocity ($v\ s$), Standard Penetration Test (SPT) blow count (N-value), or undrained shear strength in the upper 100 ft of site profile ($s\ u$) were recommended to determine the average condition of the coil class. There are six different classes A, B, C, D, E & F, and are defined as the following Table (Table 2):

Table 2. Site Classification

Site Class	*v s*	*N* **or** *Nch*	*s u*
A	> 5000 ft/sec	_	_
B	2500 to 5000 ft/sec	_	_
C	1200 to 2500 ft/sec	> 50	> 2000 psf
D	600 to 1200 ft/sec	15 to 50	1000 to 2000 psf
E	<600 ft/sec	<15 blows/ft	<1000 psf

Table note: If the *su* method is used and the *Nch* and *su* criteria differ, select the category with the softer soils (for
example, use Site Class E instead of D).

Site Coefficients

Site coefficients for the short-period range (*Fa*) and for the long-period range (*Fv*) are given in the following Tables (Table 3 & 4, respectively.) *Fa* or *Fv* are defined by the site class and spectral accelerations.

Table 3 Values of Fa as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	**Mapped Spectral Response Acceleration at Short Periods**				
	$S_s \leq 0.25$ **g**	$S_s = 0.50$ **g**	$S_s = 0.75$ **g**	$S_s = 1.00$ **g**	$S_s \geq 1.25$ **g**
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
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of *Ss*, where Ss is the spectral acceleration at 0.2 second obtained from the ground motion maps.
a Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table 4. Values of Fv as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of *S1,* where *S1* is the spectral acceleration at 1.0 second obtained from the ground motion maps.
a: Site-specific geotechnical investigation and dynamic site response analyses shall be performed

<u>Vertical Acceleration and Near Fault Effects</u>

Design for Vertical Acceleration Effects

Under Seismic Design Category D, bridges shall have at least 25% of the longitudinal top and bottom mild Reinforcement continuous over the length of the bridge superstructure to account for the effects of vertical ground motions. For precast prestressed girders, a minimum of 25% of the total equivalent mild and prestressing steel shall be in the form of continuous mild reinforcement. The continuous steel reinforcement shall be spliced with “service load” couplers capable of achieving a minimum of 125% of the nominal yield strength of the steel reinforcement. Vertical ground motions design requirements do not apply for steel girders. A case-by-case determination on the effect of vertical ground motions is required for essential and critical bridges.

Design Considerations for Near Fault Effects

For sites located within 10 km (6 miles) of an active fault, studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response. The faultnormal component of near-field (D < 10 km) motion may contain relatively long-duration velocity pulses which can cause severe nonlinear structural response, predictable only through nonlinear time-history analyses. For this case the recorded near-field horizontal components of motion needs to be transformed into principal components before modifying them to be response-spectrum-compatible.

Design and Analysis Procedure

The objective of seismic analysis is to assess displacements demands and capacities of a bridge and its individual components. So, the selection of the appropriate analysis methods is very crucial to calculate the correct demand and capacity of the structure. Generally speaking, Equivalent Static Analysis (ESA) and Linear Elastic Dynamic Analysis (EDA) are the appropriate analytical tools for estimating the displacements demands for normal bridges. Inelastic static analysis, "Pushover Analysis" is the appropriate analytical tool used to establish the displacement capacities for normal bridges assigned in the highest seismic design category (i.e. SDC D). And the Nonlinear Time History analysis is used for critical or essential bridges.

Analysis Procedures

The following Table (Table 5) illustrated the selection of analysis procedures based on the regularity type and bridge's SDC

Table 5. Selection of Analysis Procedure

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	Use Procedure 1 or 2	Use Procedure 2

Table Note: *PROCEDURE 1: Equivalent Static Analysis Method*
PROCEDURE 2: Multimode Spectral Method
PROCEDURE 3: Nonlinear Time History Method (used primarily in critical or essential bridges)
Single span bridges are not required to perform detailed seismic analysis. However, minimum seismic horizontal forces for connections and minimum seat width are required.

Minimum Seat Width

These revised recommendations (20-7/193) require minimum seat width to all SDCs. Minimum bearing support lengths as determined in this section shall be provided for the expansion ends of all girders.

For SDC A, the minimum seat width is

$$N = (4 + \Delta_{ot} + 0.2H)\left(1 + S^2/4000\right) > 12"$$

Where Δot : movement attributed to prestress shortening creep, shrinkage and thermal expansion or contraction to be considered no less than one inch per 100 feet of bridge

superstructure length between expansion joints. (in.)
H: Largest column height within the most flexible frame adjacent to the expansion joint height from top of footing to top of the column (i.e., column clear height, ft.) or equivalent column height for pile extension column (ft.).
S : angle of skew of support in degrees, measured from a line normal to the span.

For SDC B, C, & D, the minimum seat width is

$$N = \left(4 + \Delta_{ot} + 1.65\Delta_{eq}\right)\left(1 + S^2/4000\right)(\text{in.}) \geq 12$$

Δot = movement attributed to pre-stress shortening
creep, shrinkage and thermal expansion or contraction to be considered no less than one inch per 100 feet of bridge superstructure length between expansion joints (in.).
Δeq = seismic displacement demand of the long period frame on one side of the expansion joint (in.).

Concluding Remarks

The loss of life and extensive property damage suffered from recent large earthquakes, including the 1989 Loma Prieta and 1994 Northridge earthquakes, have demonstrated the earthquake vulnerability of highway bridges that were designed to existing seismic codes, and the need to provide new procedures and specifications for constructing earthquake-resistant bridges and highways. In recognition of this need, the FHWA and State Departments of Transportation (DOT), are working together to develop new seismic design provisions for highway bridges under the National Cooperative Highway Research Project 12-49 and 20-7/193. The objective of these developments is to enhance safety and economy through the development of a new LRFD specification and commentary for the seismic design of bridges. The proposed provision under NCHRP 12-49 was completed in 2001 and was revised by the NCHRP 20-7/193 project in 2006. These revised recommendations will be considered for adoption by the AASHTO Highway Subcommittee on Bridges and Structures in 2007 as guide specifications. These major changes described above were based on the three essential comments of proposed provisions (i.e. NCHRP 12-49) on 1) Design Criteria, 2) Applicability and 3) Complexity. The dual-level performance criteria based on the probabilities of exceedance (either 3% or 50%) for a normal bridge service life of 75 years, was revised to a single level (Life Safety) performance based on the probabilities of exceedance 7% for 75 years. Design analysis procedures were also simplified into three different procedures. Although displacement approach methods using inelastic static analysis method, namely "pushover analysis", was identified as an appropriate analysis method for the highest SDC – "D", however, it is not required in the recommendations. This is a displacement-based approach for analyzing dynamic response. The objective is to determine the displacement at which the earthquake-resisting elements (ERE) achieve their inelastic deformation capacity. Damage states are defined by local deformation limits, such as plastic hinge rotation, footing settlement or lift, or abutment displacement. Displacement may be limited by loss of capacity such as degradation of strength under large inelastic deformation or $p-\Delta$ effects. This displacement capacity check method is required in the design of each pier and bent. Minimum seat width for all SDCs are relatively increased to provide some safety factors in accommodating unexpected events. A completed revise recommendations will be able through AASHTO or NCHRP organizations.

References

Final Report of Recommended LRFD Guidelines for the Seismic Design of Highway Bridges by NCHRP 20-7/ Task 193.