Comparison of new AASHTO Guide Specifications for Seismic Bridge Design and updated Seismic Provisions in LRFD Bridge Design Specifications for Montana

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Abstract

The American Association of State Highway Transportation Officials (AASHTO) recently approved two significant changes in the seismic design of bridges. First is the modification of the current Load and Resistance Factor Design (LRFD) Bridge Design Specifications to increase the return period of the design earthquake from 475 years to approximately 1000 years, in addition to several updates reflecting modern practice of force based seismic design. The second development is completion of the Guide Specifications for LRFD Seismic Bridge Design. The Guide Specifications are an alternate set of provisions specifically focusing on the ductility and displacement capacity of a structure, and as such is referred to as a displacement based approach. The purpose of this paper is to compare the designs of a representative case study bridge using both the "force based" and "displacement based" specifications and assess the impact to the Montana Department of Transportation (MDT) bridge design and construction program.

Introduction

The seismic provisions in earlier versions of the LRFD Bridge Design Specifications (LRFD Specifications) were adopted in 1990 recognizing the urgent need for standardized seismic design after the Loma Prieta Earthquake of 1989. Many valuable lessons have been learned about the cause of earthquakes worldwide and the seismic performance of bridges since that time. Evaluation of damage after Loma Prieta, Northridge in 1994, Kobe 1995, Turkey 1999, Chi Chi 2000, among others have offered insight to the vulnerabilities of bridges under seismic attack and provided impetus for seismic research that can be developed and applied to a new generation of structures.

The earthquake design from previous versions of the LRFD Specifications considered a single level event with a 10% probability of exceedance in 50-years, or about a 475-year return period. The response spectrum was based on normalized elastic seismic response coefficients with 5% structure damping, and considering 3 different soil profiles. The long period portion of the spectrum was inversely proportional to $T^{2/3}$ which was intended to provide a measure of conservatism in force based seismic design. This design event applied to regular, critical, and essential structures up to 152-meters (500-feet) in length [AASHTO 2006].

The objective of the updated LRFD Specifications and new Guide Specifications is Life Safety performance for a seismic hazard corresponding to a 7% probability of exceedance in the 75-year design life of a bridge, or a 1000-year return period. Life Safety implies that the bridge has a low probability of collapse, but may sustain

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significant damage such that partial or complete replacement may be required following a design event. Higher levels of performance for critical or essential bridges that are required to be open to emergency traffic at all times, and for other types of construction such as suspension, cable-stayed, and truss bridges are not addressed in the Guide Specifications and are subject to the owners' additional requirements. LRFD Specifications maintain coverage for critical and essential bridges of regular construction. The Guide Specifications do not include recommendations for near fault effects [Imbsen, 2007].

Justification for the 1000-year return period is that the overall population of affected structures nationwide would not change drastically. Collapse prevention should be maintained when considering large historical earthquakes by taking advantage of inherent sources of conservatism in bridge components. These sources of conservatism have become more obvious based on recent observations of earthquake damage and experimental data [NCHRP 2005].

Modifications to LRFD Specifications

The LRFD Specifications have been updated to include ground motion maps using the 1000-year return period. The four Seismic Performance Zones have been re-partitioned using 1.0-second spectral accelerations at ground surface. Unique response spectra can be developed for each site using a general procedure called the "two point" method and site specific soil classifications. The rate of decay of the long period portion of the response spectrum is proportional to 1/T. Flexural resistance factor for axially loaded members (φ) has been increased for seismic applications. Relaxation of the φ-factor and the change in spectral curve shape removes some of the built in conservatism preventing strength degradations under large inelastic deformations. Thus, an explicit P-Δ check is now required. Finally, empirical support lengths have been increased to reflect new zone boundaries. The design methodology remains force based by relying on elastic design forces modified by an R factor [AASHTO 2007].

Guide Specification Philosophy

The Guide Specifications were developed using identical ground motion maps as the LRFD Specifications for a 1000-year return period and also have Life Safety as the performance goal. Development of the response spectrum will be the same with both specifications using the two point method and site specific soil classifications. There are four Seismic Design Categories (SDC) to differentiate it from the LRFD Specifications although the partitioning is identical.

The engineer is directed to choose a global design strategy or Earthquake Resisting System (ERS) and identify particular Earthquake Resisting Elements (ERE) within the complete load path for that system. Elastic methods of analysis are still used to calculate seismic displacement demands on a structure for all but the highest seismic zone, but if these demands exceed the implicit capacity of the structural elements, a non-linear static analysis (also called "pushover" analysis) must be used to further define actual demands, and is required of the highest design category. The demand and capacity evaluations are primarily related to displacement of ductile elements within the structure, rather than a force applied to those elements, and as such the methodology of this specification is often referred to as displacement based.

Case Study Bridge

To illustrate the application and comparison of the two design methodologies and assess the impact of the specifications to MDT's bridge construction program, a case study bridge was chosen representing typical construction methods, structural elements, and span lengths. Case Study Bridge is a regular 3-span continuous rolled steel girder bridge supported on one prismatic drilled shaft/column per bent, with steel pipe pile foundations and semi-integral wall at the abutments. Bridge length is 95.5-meters. Shaft and column diameter is 1.8-meters. Abutment piles are 508-mm diameter with 12.7-mm wall thickness. Geology at the site is characterized by a steeply dipping karstic limestone formation (Tertiary Sedimentary Rock) overlain by inter-bedded layers of sand, gravel and clay (Quaternary Alluvium).

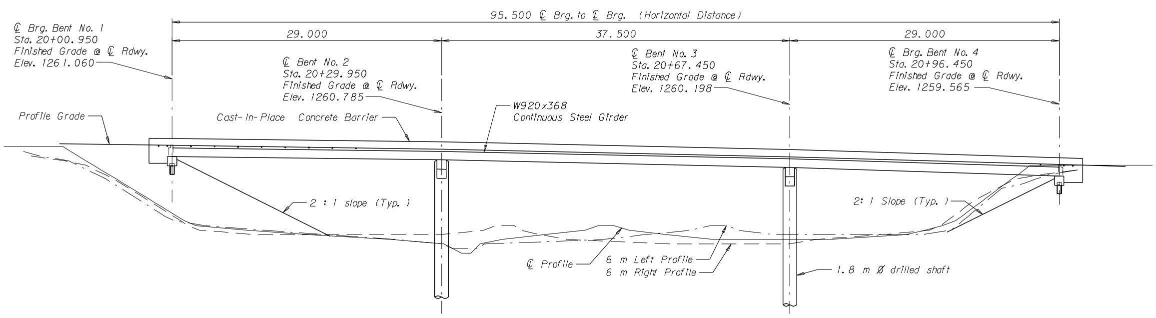


Figure 1 ELEVATION VIEW OF CASE STUDY BRIDGE

Seismic Hazard Characterization

The seismic hazard characterization is similar for both the LRFD Specifications and the Guide Specifications. Ground motion maps were prepared for AASHTO by the United States Geological Survey (USGS) for all 50 states and Puerto Rico with detailed maps of California, central Rocky Mountains, New Madrid fault region of the midwest, and South Carolina. These maps are based on USGS data used for the National Seismic Hazards Mapping Project 2002 update, except for Alaska (2006), Hawaii (1998) and Puerto Rico (2003). Companion software was developed to simplify determination of acceleration values using geographical coordinates.

To illustrate construction of the design response spectrum, consider the Case Study Bridge located in southwest Montana at geographical coordinates of 45.886° latitude and -111.411° longitude. There are no active faults within 10-kilometers of the project vicinity. Liquefaction potential is low for this site and will not be considered in this design. The two point general procedure will be used to construct the map response spectrum. The following points are found from the ground motion maps: Peak Ground Acceleration (PGA), short period spectral acceleration (S_s), and spectral acceleration for 1.0 second period (S_1).

Applying site specific coefficients to the map spectral accelerations creates the design earthquake response spectrum. These coefficients represent the soil affects on ground motion from rock to ground surface. The site may be categorized using one of three soil parameters: average shear wave velocity υ_s; average Standard Penetration Test blow counts N; and the average undrained shear strength s_u. There are six different soil types and profiles to choose from.

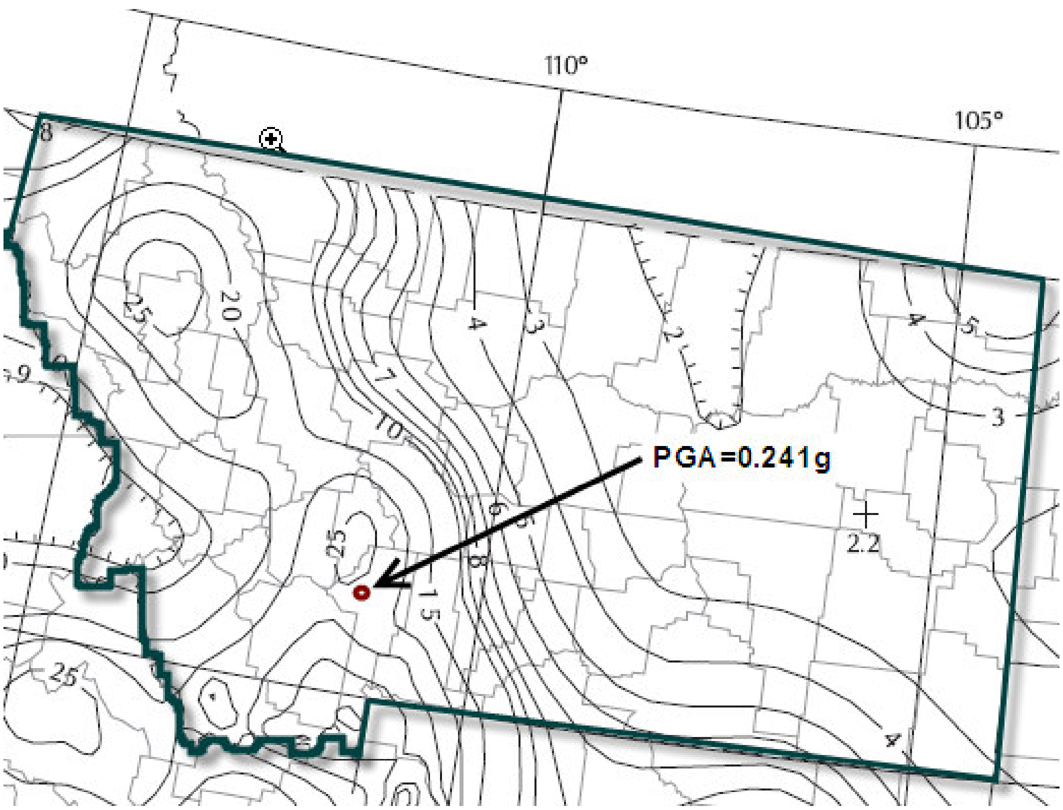


Figure 2 PEAK HORIZONTAL GROUND ACCELERATION

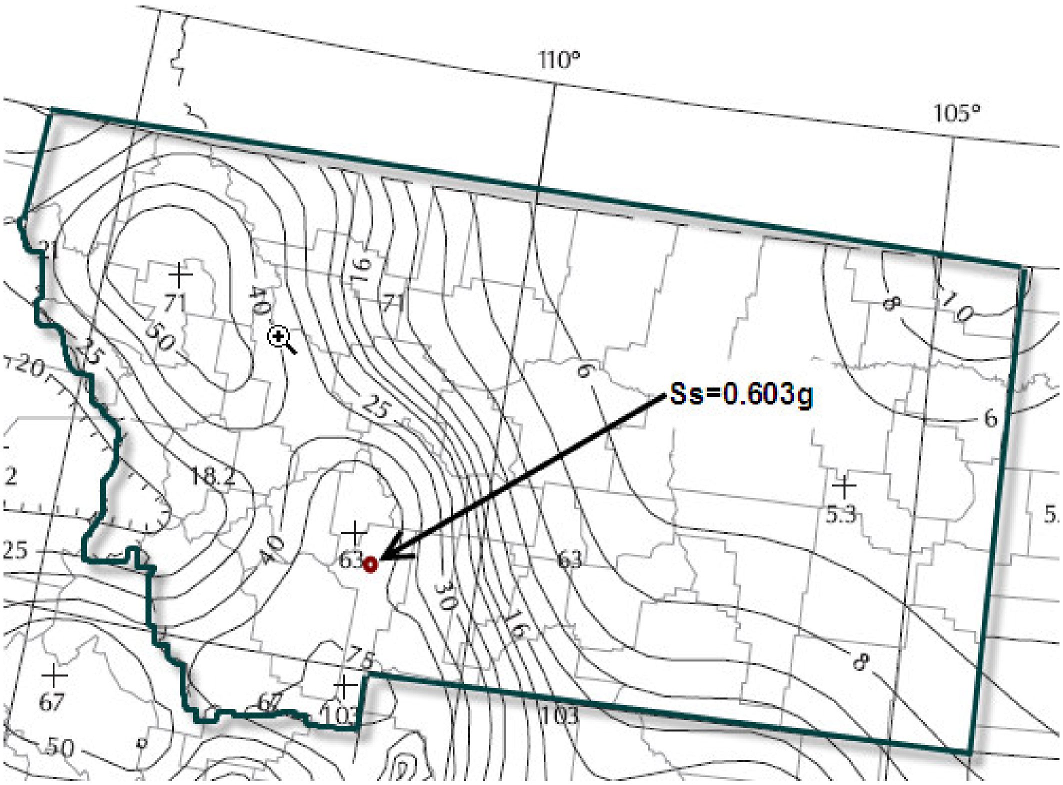


Figure 3 HORIZONTAL RESPONSE SPECTRAL ACCELERATION AT 0.2 SECOND PERIOD

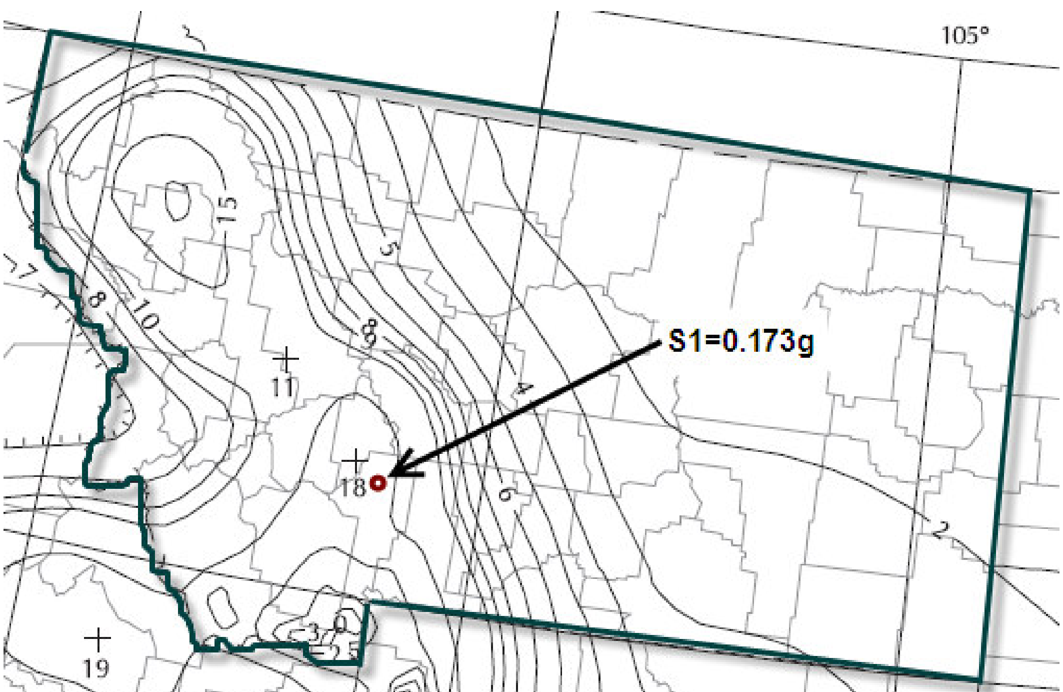


Figure 4 HORIZONTAL RESPONSE SPECTRAL ACCELERATION AT 1.0 SECOND PERIOD

Data from a representative boring log for the Case Study Bridge is shown in Table 1. The soil layers above the bedrock have an average N value of about 12 blows per 0.3m (12 blows/ft). The bedrock layer has average N value greater than 50 blows/0.3m (50 blows/ft). The drilled shafts will extend 2-meters into the bedrock. A wedge of compacted backfill with Class C or D soil characteristics will be placed at each abutment. Soil layers 1 through 6 could be characterized as a Site Class E soil profile based on the average N value, while the limestone layer 7 could be a Class B or C rock. Considering the relatively shallow depth of softer soils, the embedment of the drilled shaft into a massive limestone formation, and the influence of engineered backfill at the abutments, a reasonably conservative estimate of the site effects could be made by choosing Site Class D soils from Table 2. This should be verified by in-depth geotechnical engineering analysis, but can be used as a starting point to characterize the seismic hazard. The site coefficients are then interpolated from Table 3 for the peak ground acceleration (F_{pga}), short period range (F_a), and long period range (F_v).

Table 1 GEOTECHNICAL BORING LOG SUMMARY

Layer (i)	Soil Layer Thickness (d_i)	Blow count (N_i)	d_i / N_i	N_{avg}
1 (sand)	4.1 m	7	.585	11.7
2 (gravel)	1.6 m	28	.057	
3 (gravel)	1.6 m	16	.100	
4 (clay)	1.4 m	12	.100	
5 (clay)	1.4 m	14	.100	
6 (gravel)	1.2 m	50	.024	
7 (limestone)	2.0 m	50	.040	50
Average Standard Penetration Resistance for cohesionless soil layers, $N = \Sigma d_i / \Sigma\, d_i/N_i$				

Table 2 SITE CLASS DEFINITIONS

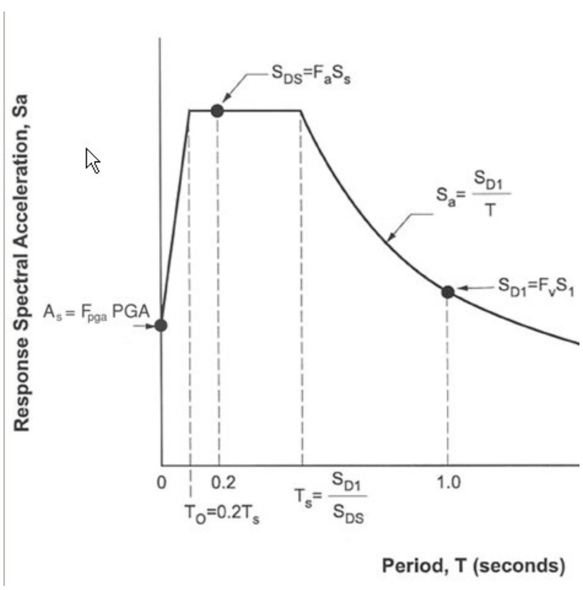
Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s >$ 5,000 ft/sec.
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/sec.
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/sec, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf.
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1,200 ft/sec, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf.
E	Soil profile with $\bar{v}_s$ < 600 ft/sec or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40 percent and $\bar{s}_u$ < 0.5 ksf.
F	Soils requiring site-specific evaluations, such as: Peats or highly organic clays (H > 10 ft of peat or highly organic clay where H = thickness of soil) Very high plasticity clays (H > 25 ft with PI > 75) Very thick soft/medium stiff clays (H >120 ft)

Table 3 SITE COEFFICIENTS

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	PGA≤ 0.10 $S_S \le 0.25$	$PGA = 0.20$ $S_S = 0.50$	$PGA = 0.30$ $S_S = 0.75$	$PGA = 0.40$ $S_S = 1.00$	$PGA \ge 0.50$ $S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4 Fpga=1.3 Fa=1.32	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

Site Class	Mapped Spectral Response Acceleration Coefficient at 1 Second Periods				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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 Fv=2.11	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

The design response spectrum is then created using the following points:



$A_s = F_{pga}*PGA = 0.34g$
$S_{DS} = F_a*S_s = 0.80g$
$S_{D1} = F_v*S_1 = 0.37g$

Figure 5 TWO-POINT RESPONSE SPECTRUM

Alternatively, the design spectrum could be created using the ground motion software by input of geographical coordinates and site class definition. The Case Study Bridge, with 1.0-second design acceleration (S_{D1}) of 0.37g would be placed into Zone 3 or SDC C.

Table 4 SEISMIC DESIGN CATEGORY AND PERFORMANCE ZONE

Value of $S_{D1} = F_v S_1$	SDC	Seismic Zone
$S_{D1} < 0.15$	A	1
$0.15 \le S_{D1} < 0.30$	B	2
$0.30 \le S_{D1} < 0.50$	C	3
$0.50 \le S_{D1}$	D	4

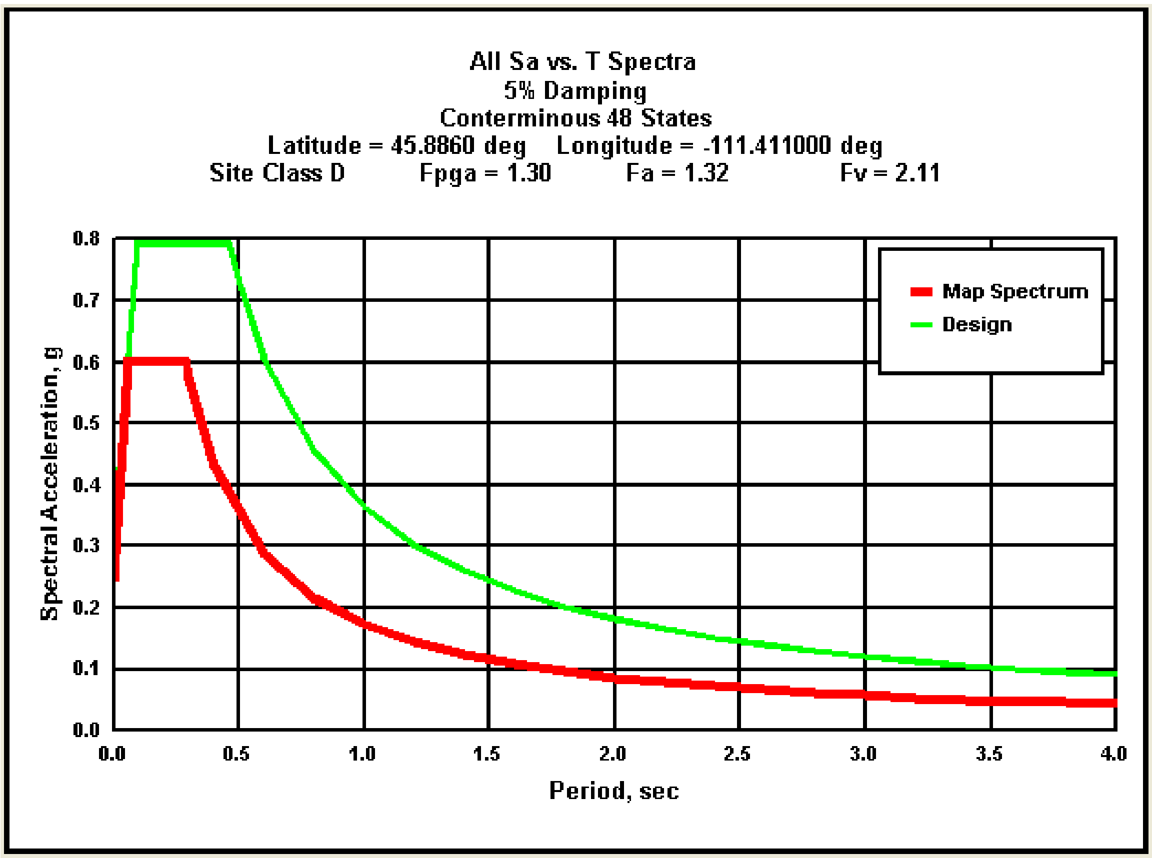


Figure 6 DESIGN SPECTRUM FROM GROUND MOTION SOFTWARE

Demand Analysis and Modeling Techniques

At this point the two design specifications diverge in philosophy. An elastic dynamic analysis will be appropriate for most situations using either specification.

However, the LRFD Specification relies on modified elastic forces to determine the force demand on the structural elements, which are then designed for sufficient strength to resist those forces. Whereas the Guide Specifications provide a simplified means to determine the displacement capacity of ductile elements which is then compared to the seismic demands from a linear elastic analysis. Designs for minimum flexural strength, shear strength, and capacity protection are also completed.

Applying the Guide Specifications to the Case Study Bridge, the first task is to consider the Earthquake Resisting System (ERS) and classify it as one of three general types: Type 1 uses a ductile substructure with an essentially elastic superstructure; Type 2 uses an essentially elastic substructure with a ductile superstructure (this applies only to steel superstructures where the ductility is achieved by yielding in the pier cross frames); and Type 3 employs a fusing mechanism or seismic isolation element between an elastic superstructure and substructure. To encourage appropriate use of these systems, a series of earthquake resisting elements (ERE) are presented in the Guide Specification which are categorized as permissible, permissible with owners approval, and not recommended.

The global design strategy for the Case Study Bridge is Type 1, where the superstructure remains elastic and drilled shaft substructure behaves inelastically and forms a plastic hinge. The EREs within this load path are transverse/longitudinal abutment soil response, and in-ground hinging of shafts. Although discouraged in the Guide Specifications, in-ground hinging was deemed acceptable for the Case Study Bridge since inelastic rotation would likely create an annular gap at ground line that could be easily observable at this site.

Figure 7 EARTHQUAKE RESISTING ELEMENTS (ERE)

Guidance is provided in each specification on the minimal and appropriate methods of modeling and analysis. However, the Guide Specification presents thorough discourse and commentary on analytical models and procedures. A summary of the requirements for each are presented in Table 5 along with the corresponding values for the Case Study Bridge. The procedures for modeling the bridge are identical using either specification: bridge components are described geometrically and equivalent stiffnesses for the substructure elements are determined. In this case, the depth to equivalent stiffness of the steel piles and drilled shafts are determined. The effective or “cracked” section properties of the prismatic shaft, which is the primary ductile element in the system, can be determined by applying the approximate techniques from the Guide Specification or completing a moment-curvature analysis. In lieu of a more rigorous analysis, the LRFD Specifications simply suggest 50% effective stiffness for the column.

LRFD Specifications lack direction on the use of passive pressure at the abutments as part of the earthquake resisting system and has often been neglected in the model. This could be a conservative approach from a force based perspective because it tends to increase the force demand on the ductile substructure elements. However, the opposite effect could occur where neglecting the abutment soil stiffness would tend to attract less force to the abutment resulting in underestimation of force on the abutment structural elements. Additionally, increased flexibility of the structure tends to lengthen the period of the structure thus shifting it onto the decaying portion of the response spectrum and decreasing the resulting force effect. Therefore, determine an enveloped response with and without soil-structure interaction at the abutments using LRFD Specifications. By contrast, approximate methods for determining equivalent abutment stiffness are presented in the Guide Specifications to be used as part of the ERS.

Support lengths are still considered the first tier method of preventing collapse of a structure. Minimum support lengths in the LRFD Specifications are calculated using the following empirical formula considering the structure length L, column height H, and skew S. Support length is increased depending on the seismic performance zone:

$$N(\text{inches}) = (8 + 0.02*L + 0.08*H)(1 + 0.000125*S^2)$$

Minimum support lengths in the Guide Specifications are determined using the same empirical formula as LRFD Specifications for SDC A, B, and C. For SDC D the empirical formula is modified to consider the structure skew S, the elastic seismic displacement demand Δ_{eq}, and modification factor R_D which increases for short period structures. Support lengths using this formula may be less than those calculated using the LRFD formula.

$$N(\text{inches}) = (4 + 1.65*\Delta_{eq}*R_D)(1+.00025*S^2) \geq 24$$

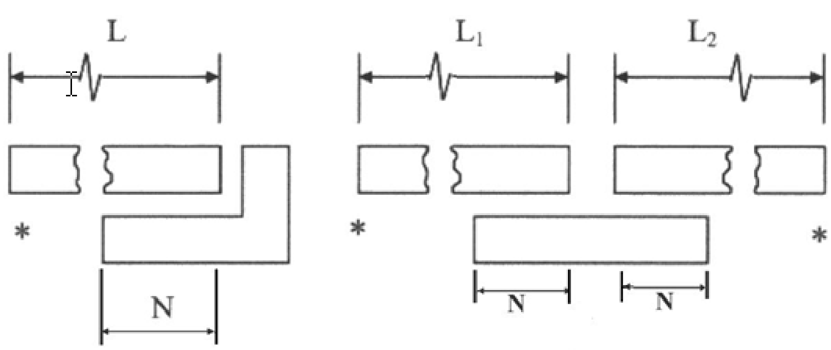


Figure 8 Support Length

Table 5 SUMMARY OF DEMAND ANALYSIS FOR CASE STUDY BRIDGE

	LRFD Specifications	Guide Specifications
Analysis Procedure	Uniform Load Elastic, Single Mode Elastic or **Multimode Elastic**	Equivalent Static or **Elastic Dynamic**
Soil – Structure Interaction at Abutments	Designed to resist seismic force envelope with and without contribution from abutment soil in both orthogonal directions.	Secant Stiffness models for passive resistance. Maximum of 70% soil resistance at abutments in both orthogonal directions part of the ERS.
Force (Flexural) Demand of Ductile Element	$M_E = M_{elastic}/R$ where R_{column}=3.0 for regular, non-essential bridge M_E = 5435 kN*m	$M_{ne} \geq 0.1 P_{trib} \frac{(H_h + 0.5 D_s)}{\Lambda}$ where P_{trib} = column axial load D_s = depth of superstructure Λ = fixity factor of column H_h = height of column M_{ne} = 5430 kN*m
Shear Demand	$V_{overstrength}$ = 905 kN	$V_{overstrength}$=1005 kN
Connection Strength – Capacity Protected	$V_E = V_{elastic}/R$ where $R_{connection}$ = 1.0 at pier V_E = 1325 kN	$V_D = 0.4*P_{trib}$, or $V_{elastic}$, $V_{overstrength}$ $V_D = .4*P_{trib}$ = 1575 kN
Displacement Demand	Local displacement demand Δ_D = 140mm	Local displacement demand Δ_D = 100mm
Displacement Ductility Demand	No specified requirements. R factor nearly approximates μ	Ductility Demand SDC C μ = 3 Ductility Design Target μ = 4
Foundation Flexibility	Model developed to maximize force demands on substructure elements.	Model developed to approximate displacement demands on structure. Foundation flexibility must be considered in design.

<u>Capacity Analysis and Detailing Requirements</u>

The structural capacity determination using the LRFD method essentially examines the flexural and shear capacity of the section using over-strength factors for expected material properties. Basic guidance on over-strength factors for concrete and reinforcing steel are given in Section 3 Appendix B of the LRFD specifications [AASHTO 2006]. The resistance factor φ for flexural capacity of axially loaded members is determined based on the seismic zone and approaches a value of 0.9, similar to that for flexural members, for zones 3 or higher. Secondary moments caused by P-Δ effects are limited to 25% of the factored nominal resistance of the section. Maximum longitudinal reinforcing limit has been decreased from 6% to 4% of the gross section of a column, which is intended to encourage higher member ductility [NCHRP 2003]. Transverse reinforcement requirements have not changed.

The Guide Specifications differ from LRFD Specifications in the capacity analysis of the structure by evaluating the displacement capacity of the members. Expected material strengths are determined using well researched and proven models such as monotonic tensile stress-strain model for steel reinforcing, and Mander's stress-strain model for confined concrete.   This has an impact on the capacity checks of the structure such as minimum lateral strength, transverse reinforcement requirements, plastic flexural capacity, curvature and displacement ductility of a column element.

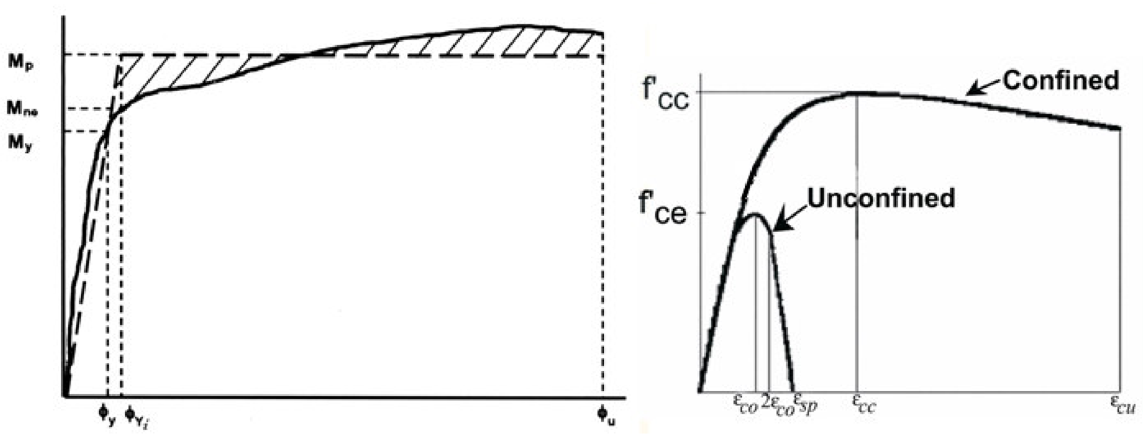


Figure 9 CAPACITY DESIGN MODELS FOR REINFORCED CONCRETE

The Case Study Bridge seismic capacity determination is based on the longitudinal and transverse reinforcement required to satisfy minimum steel ratios and resist Strength Load Combination effects. Slight differences in the results of the two analyses are noted due to the expected material strengths and over-strength factors suggested, as well as minimum reinforcing limits. The results are summarized and compared in Table 6.

Detailing procedures for joints and end regions of members is much more explicit in the Guide Specifications. The use of strut and tie models are applied to the Case Study Bridge for design of the column cap. For other framed connections, additional joint shear reinforcing is prescribed.

Table 6 SUMMARY OF CAPACITY AND DETAILING REQUIREMENTS OF CASE STUDY BRIDGE

	LRFD Specifications	Guide Specifications
Expected Material Properties	f'_{ce} = 27 MPa f_{ye} = 517 MPa ε_{cc} = 0.01 (recommended)	f'_{ce} = 34 MPa f_{ye} = 455 MPa ε_{cc} = 0.012 (Mander's Model)
Longitudinal Reinforcement	$.01\ A_g \leq A_s \leq .04\ A_g$ Used $A_s = 0.01\ A_g$	$.007\ A_g \leq A_s \leq .04\ A_g$ Used $A_s = 0.009\ A_g$
Longitudinal Splice Location	Center of Column	Outside plastic hinge region, Plastic hinge length L_p=.08*H_{column} + $ø_{column}$ = 2800mm
Transverse Reinforcing	Min. reinf. ratio = .006 A_{sp} = #16 @ 75 mm	Min. reinforcement ratio= .005 A_{sp} = #16 @ 100 mm
Transverse Detailing	Hoops with seismic hooks, welded or mechanically spliced spirals	Butt welded hoops or continuous spirals in hinge zone
Flexural Capacity (Expected Strength)	φM_n = 8700 kN*m (φ=0.9) $M_{overstrength}$ = 13100 kN*m	φM_n = 9900 kN*m (φ=1.0) $M_{overstrength}$ = 13190 kN*m
Local Displacement Capacity	Procedure not provided in specifications. Results similar using Guide Spec.	$\Delta_C^L = 0.12H_o\left(-2.32\ln(x)-1.22\right) \geq 0.12H_o$ = 205 mm where H_o is column height
Support Length	$N_{support\ length}$ =150% *$N_{emperical}$ $N_{support\ length}$ = 680mm	$N_{support}$ = 680mm ,SDC C criteria $N_{support}$ = 610mm ,SDC D criteria
Displacement Ductility Provided	μ = 4.4	μ = 4.6

Conclusions

This Case Study Bridge is representative of common construction in Montana. Application of either the LRFD Specifications or the Guide Specifications results in a structural system that is capable of resisting seismic loads in a ductile manner. In this case, simply meeting the minimum seismic design and detailing requirements for the ductile column results in more than adequate flexural and shear capacity for the Extreme Event with very little increase in material required for Strength Load Combinations as well. Targeted limited ductility performance was slightly exceeded, indicating there is sufficient reserve ductility capacity than is demanded from the analyses, possibly resulting in less sustained damage. Computational effort required to complete the designs using either specification was similar. The Guide Specification design appears to provide slightly higher ductility and better economy than the LRFD Specifications.

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