American Railway Engineering and Maintenance of Way Association

Letter Ballot 09-22-02

Ballot Contents:

The letter ballot is presented as follows:

- Proposed Revisions begin on page 3:
  - Deleted text noted by ~~strikethrough~~.
  - Added text shown in red.
  - Moved text shown in ~~green~~ and green
- Incorporation of Proposed Revisions (i.e. "track changes" accepted) begin on page 35

For ease of reference during balloting, articles are presented first, followed immediately by their respective commentary.

Proposed revisions to article numbering and reference lists may require cross-reference updates to other articles within the chapter. For balloting purposes, current article and reference numbering are assumed, except where directly modified by this ballot. Future cross-reference updates would be anticipated upon final ballot approval.

1.3.2.3 ~~Base~~ Acceleration Coefficient Maps

~~Several base~~ A~~a~~cceleration coefficient maps are provided in this Article to help define the seismic hazard. Figures 9-1-1 through 9-1-3 show peak ground~~, short-period (0.2 second) and long-period (1.0 second)~~ accelerations in the United States for return periods of 100 years, 475 years and 2475 years. These maps are mainly for illustration purposes and more accurate acceleration coefficients may be determined using web-based interactive tools found on the United States Geological Survey (USGS) website. Acceleration coefficients for sites located in Canada may be determined using the tools found on the ~~Geological Survey of Canda (GSC)~~ Natural Resources Canada (NRC) website. Other sources or site-specific procedures may be used to define the base accelerations as long as they are based on accepted methods.

The USGS tools allow direct determination of acceleration for any return period. The NRC tools provide accelerations for 10 discrete probabilities of exceedance. The NRC probabilities of exceedance correspond to return periods shown in Table 9-1-6. ~~Base a~~Accelerations ~~coefficients with~~for return periods other than ~~100 years, 475 years or 2475 years~~those shown may be ~~determined based on the following formulas:~~determined from log-log (base 10) interpolation/extrapolation.

Table 9-1-6. Return Periods for NRC Probabilities of Exceedance

Probability of exceedance in 50 years	Return period in years
2%	2475
3%	1642
4%	1225
5%	975
7%	689
10%	475
14%	332
20%	225
30%	141
40%	98

- ~~Peak ground acceleration for return period, R, less than 475 years~~

$$\cancel{PGA_R = PGA_{475}\left(\frac{R}{475}\right)^n} \qquad \cancel{n = \frac{\ln\left(\frac{PGA_{100}}{PGA_{475}}\right)}{-1.558}}$$

- ~~Peak ground acceleration for return period, R, between 475 years and 2475 years~~

~~$PGA_R = e^n$~~

~~$n = \ln(PGA_{475}) + [\ln(PGA_{2475}) - \ln(PGA_{475})] \times [0.606 \times \ln(R) - 3.73]$~~

~~PGA_R = Base peak ground acceleration coefficient for return period = R~~

~~PGA_{100} = Base peak ground acceleration coefficient for return period = 100 years~~

~~PGA_{475} = Base peak ground acceleration coefficient for return period = 475 years~~

~~PGA_{2475} = Base peak ground acceleration coefficient for return period = 2475 years~~

- ~~Short-period (S_S) and long-period (S_1) spectral response accelerations for return period, R, may be determined based on the formulas above by substituting the appropriate variables (S_S or S_1) for PGA.~~

~~$S_{S,R}$ = Base short-period (0.2 second) spectral response acceleration coefficient for return period = R~~

~~$S_{S,100}$ = Base short-period (0.2 second) spectral response acceleration coefficient for return period = 100 years~~

~~$S_{S,475}$ = Base short-period (0.2 second) spectral response acceleration coefficient for return period = 475 years~~

~~$S_{S,2475}$ = Base short-period (0.2 second) spectral response acceleration coefficient for return period = 2475 years~~

~~$S_{1,R}$ = Base long-period (1.0 second) spectral response acceleration coefficient for return period = R~~

~~$S_{1,100}$ = Base long-period (1.0 second) spectral response acceleration coefficient for return period = 100 years~~

~~$S_{1,475}$ = Base long-period (1.0 second) spectral response acceleration coefficient for return period = 475 years~~

~~$S_{1,2475}$ = Base long-period (1.0 second) spectral response acceleration coefficient for return period = 2475 years~~

C - 1.3.2.3 ~~Base~~ Acceleration Coefficient Maps

Acceleration coefficient maps reflect the seismic hazard at a site. They account for both maximum ground motion intensity expected and frequency of occurrence. The maps give ~~ground~~ acceleration levels with a uniform probability of being exceeded in all areas of the country. The steps involved in the development of these maps include: (1) the definition of the nature and location of earthquake sources, (2) magnitude-frequency relationships for the source, (3) attenuation of ground motion with distance from the source, and (4) determination of ground motion parameters at the site having the required probability of exceedance.

The ~~base~~ peak ground acceleration maps for return periods of 100 years, 475 years and 2475 years in the United States were prepared by the United States Geological Society (USGS) for AREMA. These maps are included mainly for illustrative purposes. Procedures for determining design accelerations for sites located in the United States and Canada are described in the following paragraphs.

Accelerations for sites in the United States may be ~~estimated from the maps or, more accurately,~~ determined by using the interactive tools found on the USGS website at ~~http://earthquake.usgs.gov/hazards/~~ https://earthquake.usgs.gov/nshmp/. Determination of accelerations for sites in Canada will require the use of a web-based hazard calculator found on the ~~Geological Survey of~~Natural Resources Canada (~~GSC~~NRC) website at ~~http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/~~ https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php. Example procedures for each website are shown below. The acceleration values shown are for example purposes only and should not be used for design.

Procedure for sites in the United States

~~b.~~ Navigate to ~~http://earthquake.usgs.gov/hazards/~~https://earthquake.usgs.gov/nshmp/.

b. Select the ~~''Seismic Hazard Maps and Site-Specific Data~~Hazard Curves (static)' link~~, then select the 'Unified Hazard Tool' link under the Hazard Tools heading~~. A page will appear that requires the user to select a~~n~~ ~~Edition~~Model, Location ~~and,~~ Site Class and Return Period.

c. For this example, select 'Static Hazard Curves for the 2018 Conterminous U.S. ~~2008 (v3.2.x)~~' from the dropdown menu for the ~~Edition~~Model.

d. Type in site location information (for this example Latitude = 33.06277, Longitude = -115.759).

e. ~~The default~~Select Site Class (for this example use BC). ~~is the Site Class B/C boundary which corresponds to Site Class B in the Site Factor Tables shown in Article 1.4.4.1.2.~~

f. ~~Add custom return periods as needed~~Enter return period (for this example a return period = 100 years was ~~added~~used).

g. Select ~~''Compute Hazard Curve~~Plot~~''~~. Hazard Curves and a Uniform Hazard Response Spectrum will appear. Acceleration values may be determined ~~graphically by scrolling across the vertices of the response spectra curves~~by selecting the 'Response Spectrum Data' tab. Acceleration values for this example are as follows:

PGA	~40% in 50 yrs	0.~~2498~~ 2766 (g)
PGA	10% in 50 yrs	0.~~4660~~ 5408 (g)
PGA	2% in 50 yrs	0.~~7672~~ 8976 (g)
~~0.2 sec~~	~~~40% in 50 yrs~~	~~0.5974 (g)~~
~~0.2 sec~~	~~10% in 50 yrs~~	~~1.1468 (g)~~
~~0.2 sec~~	~~2% in 50 yrs~~	~~1.9356 (g)~~
~~1.0 sec~~	~~~40% in 50 yrs~~	~~0.1696 (g)~~
~~1.0 sec~~	~~10% in 50 yrs~~	~~0.3384 (g)~~
~~1.0 sec~~	~~2% in 50 yrs~~	~~0.5757 (g)~~

Lat: 33.06277, Lon: -115.759

Notes:

~40% in 50 yrs = 100-year Return Period

2% in 50 yrs = 2475-year Return Period

10% in 50 yrs = 475-year Return Period

~~~40% in 50 yrs = 100-year Return Period~~

Procedure for sites in Canada

a. Navigate to https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php� ~~http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/~~

b. Locate ~~the 'Hazard Maps and Calculations'~~ and select '~~Hazard Calculators – Determine seismic hazard at your site~~2020 National Building Code of Canada Seismic Hazard Tool'

c. ~~Select 'Get 2010 hazard values' from available selections~~Enter the shear wave velocity (V_{s30}) or select the site class (X_s). For this example, select site class C.

d. Enter the Latitude and Longitude of the site under consideration (for this example Latitude = 48.4133, Longitude = -71.0666) and select Set coordinates.

~~e. Select the 'Number of closest points for interpolation' from the pull-down menu (for this example 15 points was selected, it is recommended to run the calculation on all 3 available options and select the highest acceleration values for design)~~

~~f. Enter additional optional information as desired (for this example no additional information was entered)~~

e. Click on '~~CALCULATE~~Obtain Seismic Hazard Values'

The page will reload after the calculation is complete. Scroll down to the acceleration values which will appear similar to this:
~~~

2%/50 years (0.000404 per annum) probability

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
~~0.578~~1.11 g	0.~~323~~656 g	0.~~153~~347 g	0.~~052~~155 g	0.~~311~~592 g

~~40~~5%/50 years (0.001 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.~~085~~645 g	0.~~044~~37 g	0.~~019~~187 g	0.~~007~~081 g	0.~~035~~351 g

10%/50 years (0.0021 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.~~241~~401 g	0.~~135~~229 g	0.~~063~~112 g	0.~~022~~0472 g	0.~~107~~145 g

Additional return periods are available by selecting the “Additional Values” tab and selecting a probability value (% exceedance in 50 years) from the drop-down menu. For this example, 40% was selected representing a 100 year return period.

~~5~~40%/50 years (0.~~0~~01 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.~~348~~119 g	0.~~206~~0664 g	0.~~099~~0301 g	0.~~033~~0117 g	0.~~184~~0636 g

Notes:

2%/50 years = 2475-year Return Period

5%/50 years = 975-year Return Period

10%/50 years = 475-year Return Period

40%/50 years = ~100-year Return Period

Future earthquakes and earthquake research will continue to improve the overall understanding of the seismic hazard and will result in revisions to the acceleration maps. The ~~2008~~ 2018 edition of the USGS maps and the ~~2010~~ 2020 edition of the ~~GSC~~ NRC maps were used in the examples above. More recent maps, maps from different sources, or site-specific procedures may be used as long as they are based on accepted methods and are consistent with the site ~~factors~~ conditions and response spectra equations in Article 1.4.4.

~~Formulas are included to determine base a~~Accelerations for return periods other than those shown on the NRC maps may be estimated using log-log (base 10) interpolation/extrapolation between listed return periods. ~~These~~ This ~~formulas are~~ approach is based on the procedure shown in Article ~~2.6.1.3~~A-4.1.8.4(6) of Reference 19. ~~Reference 13. The FEMA 273 formulas were simplified for use with the AREMA base~~

~~acceleration maps. The FEMA 273 formula for return periods less than 475 years has an exponent that is based on the acceleration level and site location. This exponent can be determined more directly using the accelerations for return periods of 100 and 475 years. Section C2.6.1.3 of Reference 14 indicates that the acceleration-return period curves are nearly linear on a log-log plot between return periods of 475 years and 2475 years, therefore a single formula is used in this range. Example peak ground acceleration vs. return period curves, developed using the formulas shown in this Article, are shown in Figure 9-C-1 for various cities throughout the United States. These curves were developed for example purposes only using specific latitude and longitude values and should not be used for design.~~

For example, using the values for site location at Latitude = 48.4133, Longitude = -71.0666 and return period of 400 years, the peak ground acceleration value is determined as shown below.

PGA(475) = 0.222 from NRC website

PGA(332) = 0.173 from NRC website

Log(PGA (400)) = log(0.173) + (log(0.222)-log(0.173)) x (log(400)-log(332))/(log(475)-log(332))

Log(PGA(400))=-0.7056

PGA(400) =10^0.7056 = 0.197

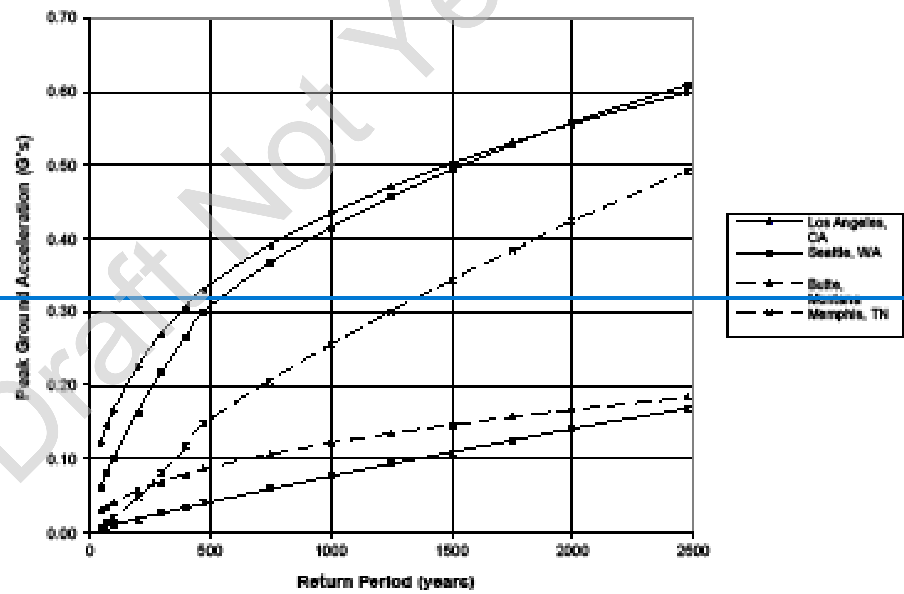


Figure 9-C-1. Peak Ground Acceleration vs. Return Period

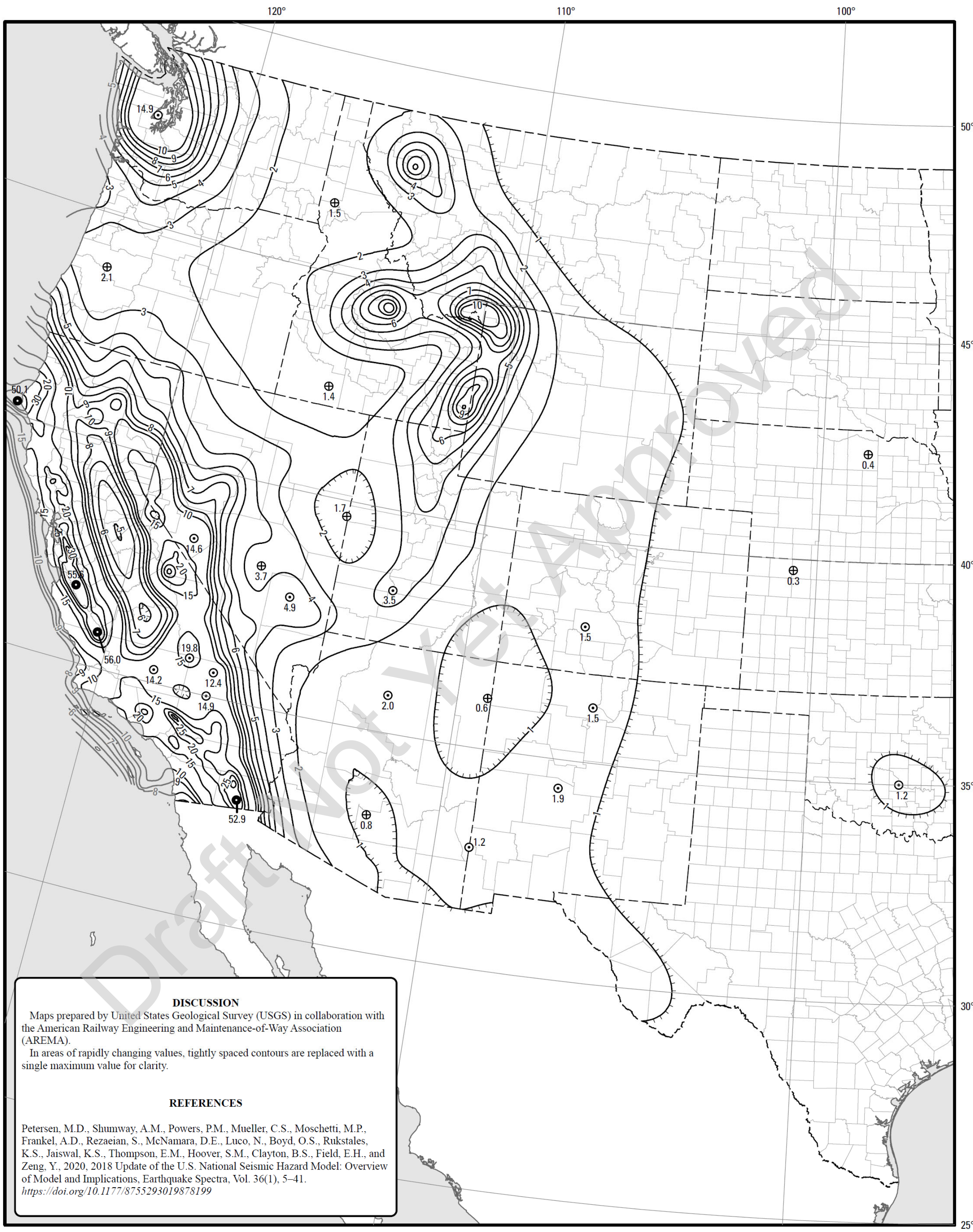


Figure 9-1-1. 100-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States

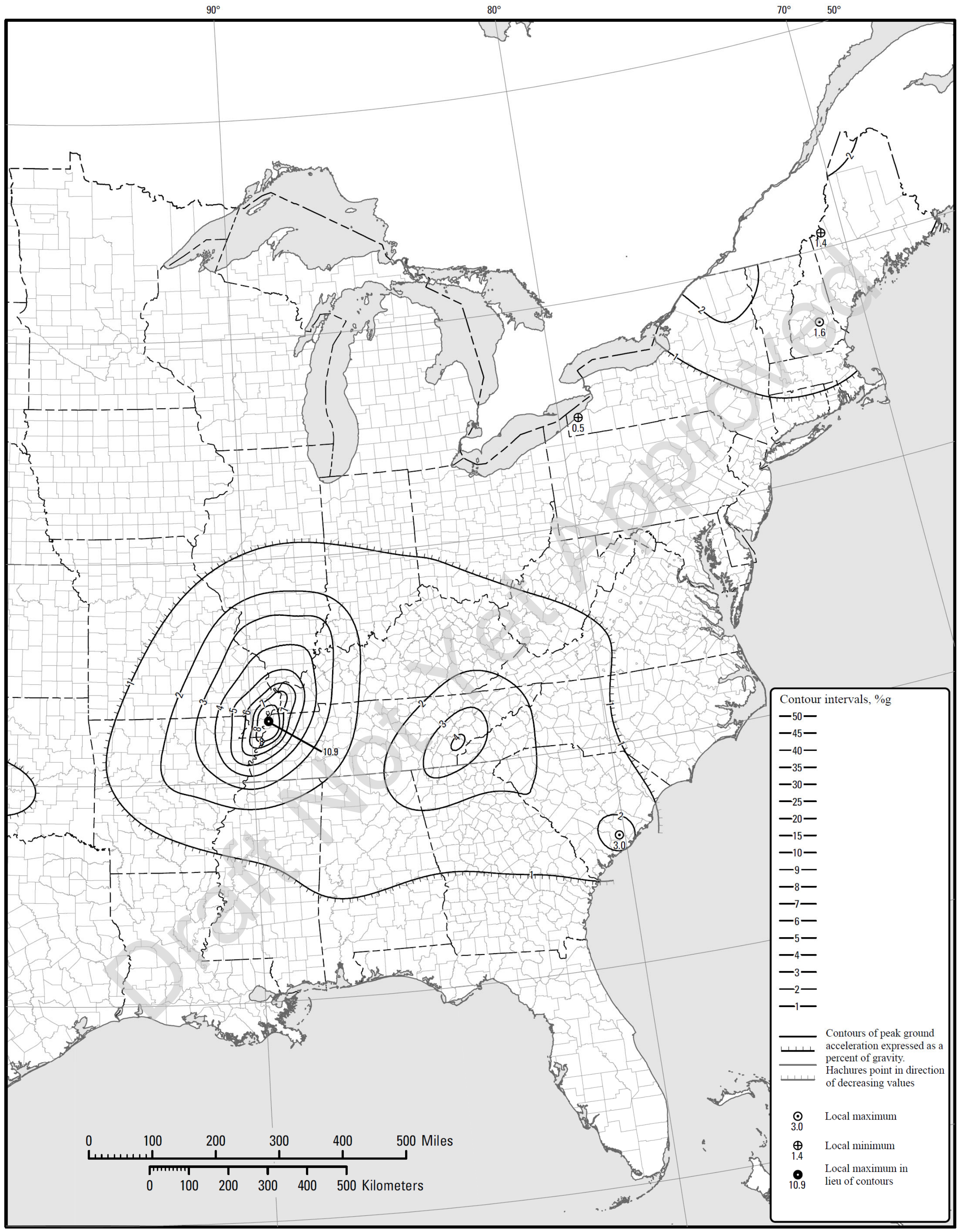


Figure 9-1-1. 100-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

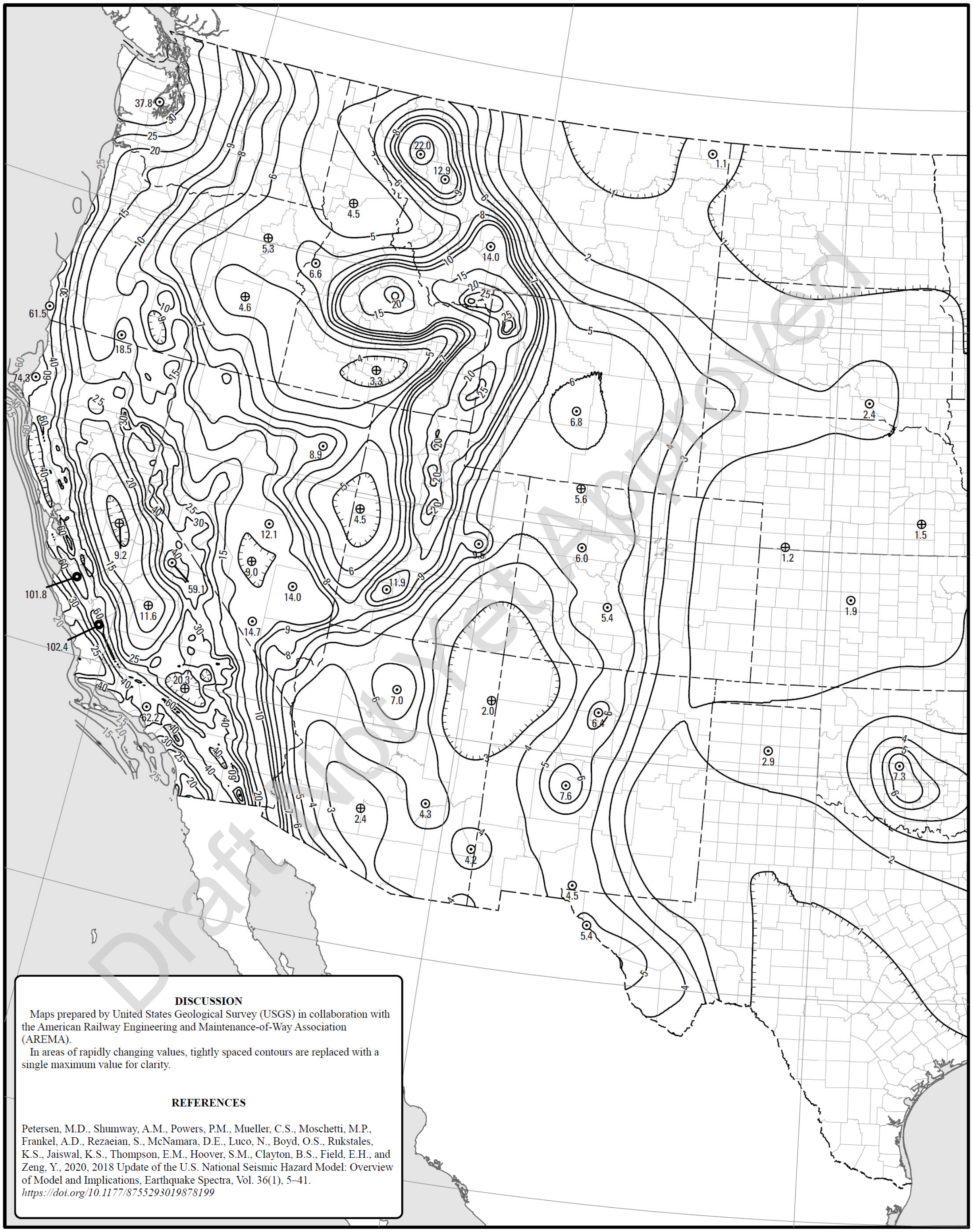


Figure 9-1-2. 475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States

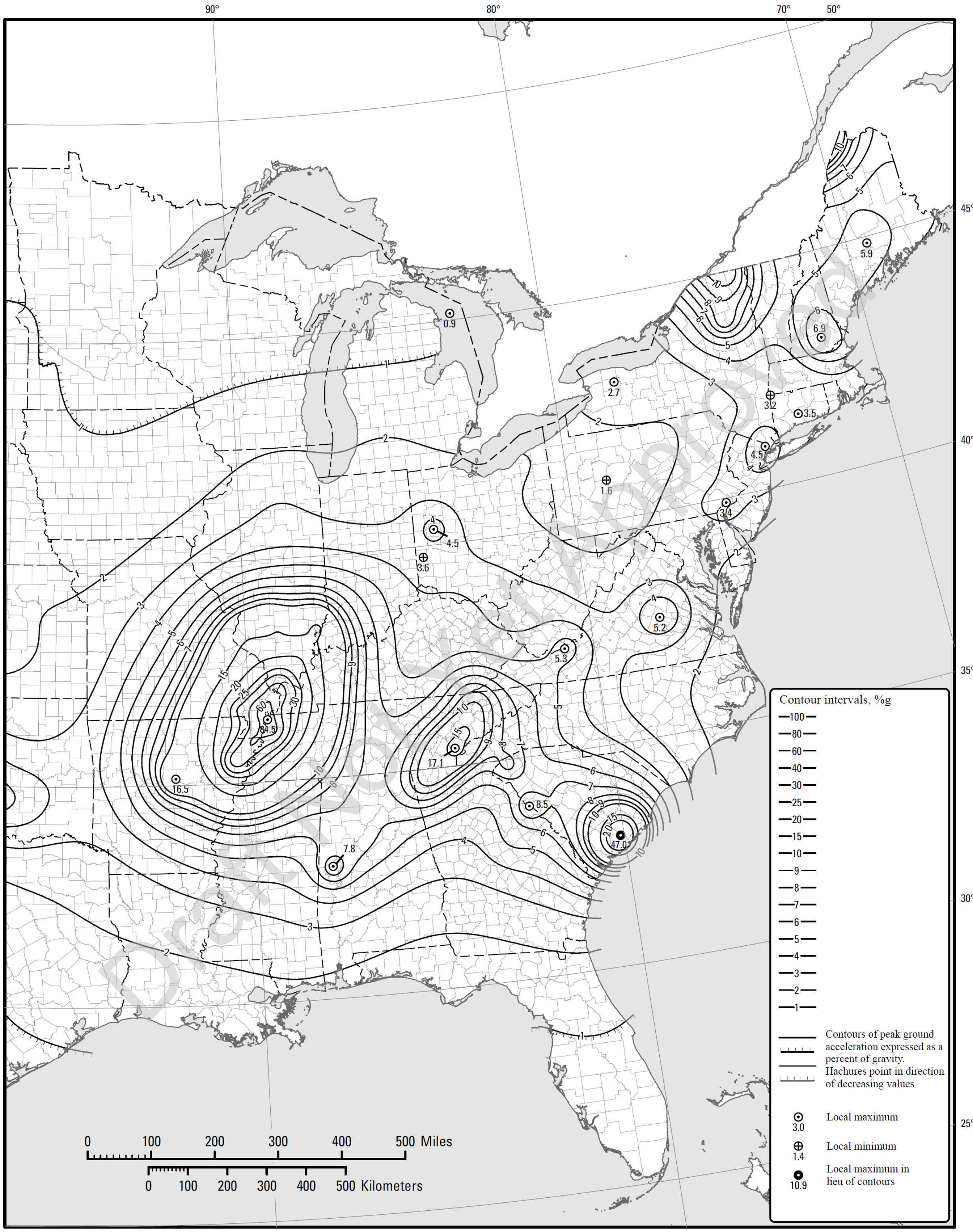


Figure 9-1-2. 475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

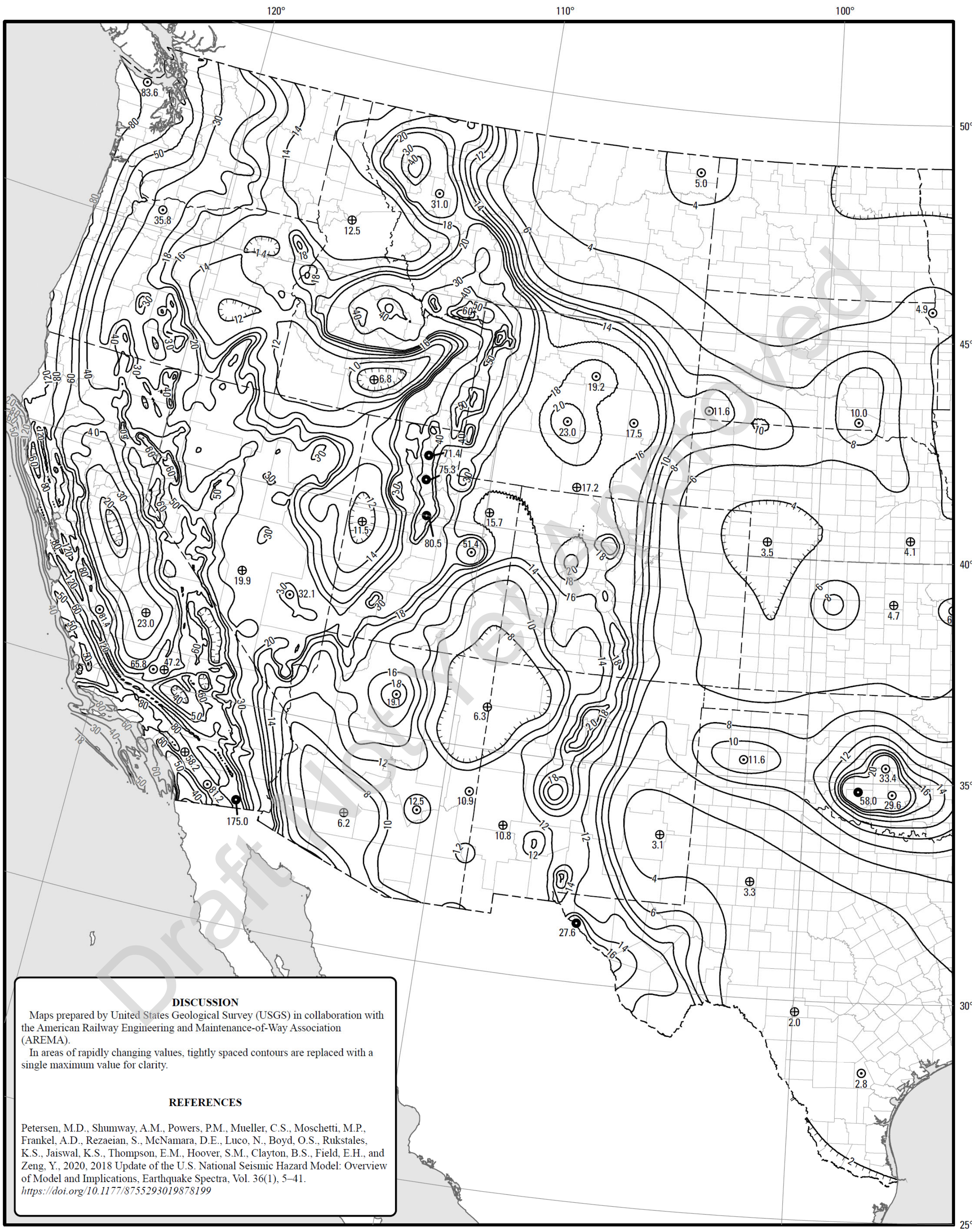


Figure 9-1-3. 2475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration — United States

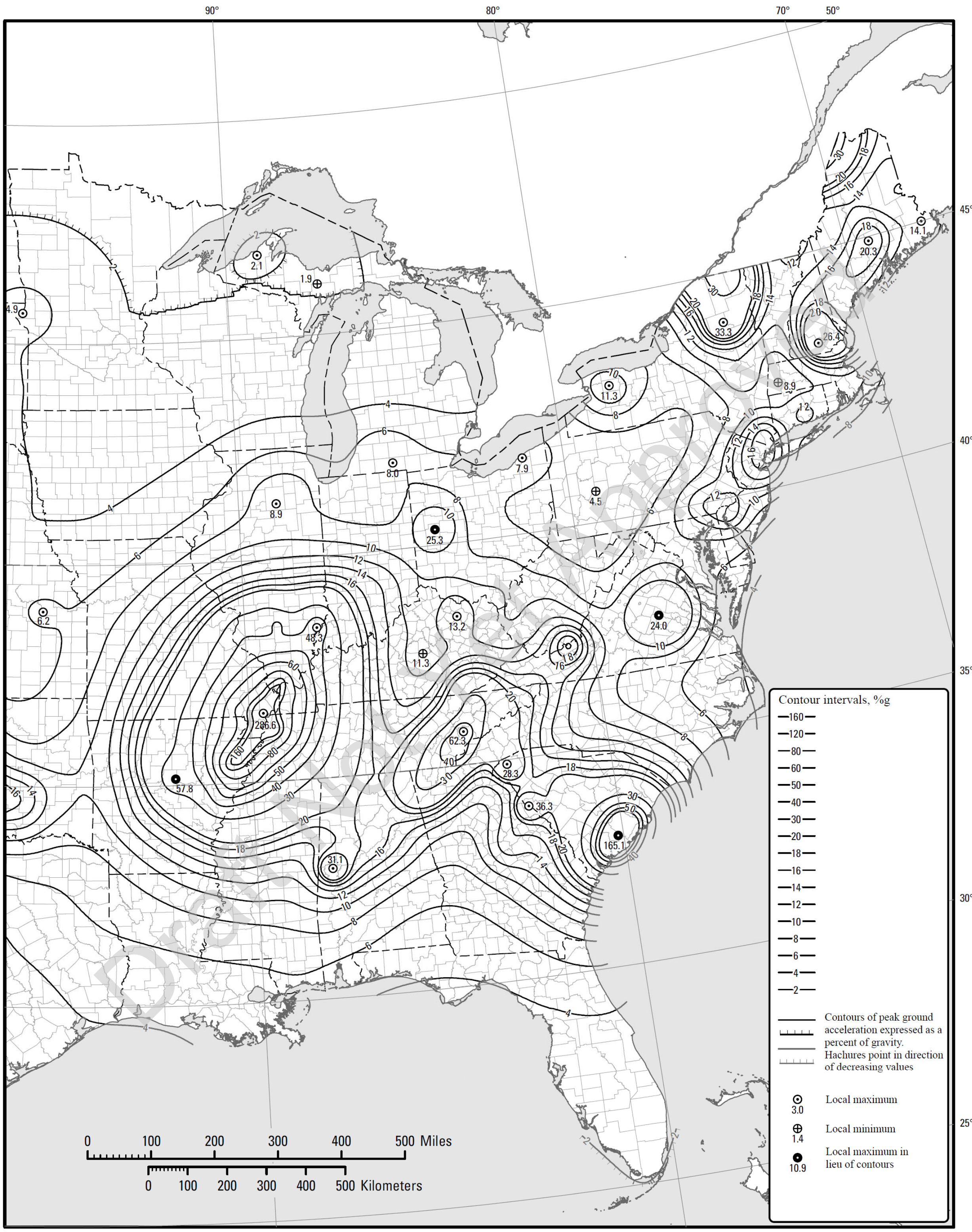


Figure 9-1-3. 2475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

1.4.4.1 Site Effects

The effects of site conditions on the response spectrum shall be determined ~~according to Article 1.4.4.1.1 and Article 1.4.4.1.2~~ based on the foundation soil characteristics.

~~1.4.4.1.1~~ Site Classification

~~1.4.4.1.1 Site Class~~

A site shall be classified as Site Class A, B, BC, C, CD, D, DE, E, or F ~~as A through F~~ in accordance with Site Cl. Sites shall be classified by their ~~stiffness~~ time-weighted average shear wave velocity in the upper 100 feet (30 m) of the soil profile, which is defined by the $\bar{v}_s$ parameter. The $\bar{v}_s$ parameter is calculated as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where

$\bar{v}_s$ = the time-weighted average shear wave velocity for the upper 100 feet (30 m) of the soil profile

d_i = the thickness of any soil or rock layer between 0 and 100 feet (30 m);

v_{si} = the shear wave velocity in feet per second (m/s); and

the summation $\sum_{i=1}^{n} d_i$ is equal to 100 feet (30 m).

Table 9-1-6. Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, v_s > 5,000 ft/s (1,5~~00~~24 m/s)
B	Rock with 3~~2~~,~~5~~000 ft/s (~~760~~ 915 m/s) < $v_s \le$ 5,000 ft/s (1,5~~00~~24 m/s)
BC	Rock with 2,100 ft/s (640 m/s) < v_s < 3,000 ft/s (915 m/s)
C	Very dense or hard soil and soft rock with 1,4~~20~~50 ft/s (~~360~~ 44~~0~~2 m/s) < $v_s \le$ 2,1~~5~~00 ft/s (~~760~~ 640 m/s)~~, or with either N > 50 blows/ft (blows/0.3 m), or s_u > 2.0 ksf (100 kPa)~~
CD	Dense or very stiff soil with 1,000 ft/s (30~~0~~5 m/s) < v_s < 1,450 ft/s (44~~0~~2 m/s)
D	Medium dense or stiff ~~Stiff~~ soil with 7~~6~~00 ft/s (~~180~~ 21~~0~~3 m/s) $\le v_s \le$ 1,0~~2~~00 ft/s (~~360~~ 305~~0~~ m/s)~~, or with either 15 ≤ N ≤ 50 blows/ft (blows/0.3 m), or 1.0 ksf (50 kPa) ≤ s_u ≤ 2.0 ksf (100 kPa)~~
DE	Loose or medium stiff soil with 500 ft/s (15~~0~~2 m/s) < v_s < 700 ft/s (21~~0~~3 m/s)
E	Very loose or ~~S~~soft soil with v_s < 5~~6~~00 ft/s (~~180~~ 15~~0~~2 m/s), or with ~~either N < 15 blows/ft (blows/0.3 m), or s_u < 1.0 ksf (50 kPa), or any profile with more than 10 feet (3 m) of soft clay defined as soil with PI > 20, w ≥ 40 percent and s_u < 0.5 ksf (25 kPa)~~
F	Soils requiring site-specific evaluations, such as: • Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. • Peats or highly organic clays (H > 10 feet (3 m) of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 feet (7.6 m) with PI > 75) • Very ~~thick soft~~thick, soft/medium stiff clays (H > 120 feet (37~~6~~ m) with s_u < 1.0 ksf (48~~50~~ k Pa)

~~v_s = average shear wave velocity for the upper 100 feet (30 m) of the soil profile~~

Estimation of Shear Wave Velocity Profiles

The $\bar{v}_s$ parameter should be derived from the measured shear wave velocity profile or, if shear wave velocity measurements are not available, from appropriate correlations with standard penetration test (SPT) blow counts, cone penetration test (CPT) resistance measurements, or soil strength and index properties from laboratory testing. Correlations may be based on site-specific relationships or published equations. See the commentary for guidance on selecting appropriate correlations.

If shear wave velocity measurements are not available for the site, the site class should be derived for $\bar{v}_S$, $1.3\bar{v}_S$, and $\bar{v}_S/1.3$, to account for uncertainities associated with estimating the shear wave velocity profile from SPT, CPT, or lab-based correlations. Ground motion parameters should then be developed for design using the most critical of the site classes resulting from $\bar{v}_S$, $1.3\bar{v}_S$, and $\bar{v}_S/1.3$ at each period in the multi-period response spectra.

~~N = average Standard Penetration Test (SPT) blow count (blows/ft (blows/0.3 m)) for the upper 100 feet (30 m) of the soil profile~~

~~s_u = average undrained shear strength in ksf (kPa) for the upper 100 feet (30 m) of the soil profile~~

~~PI = plasticity index~~

~~w = moisture content~~

~~1.4.4.1.2 Site Factors~~

~~Site factors shall be determined from Table 9-1-7 through Table 9-1-11 based on the Site Class determined from Table 9-1-6, and the values of the acceleration coefficients, and the location of the site.~~ Note that site factors differ between the ~~Separate tables are provided for~~ United States Geological Survey (USGS) and Geological Survey of Canada (GSC) based accelerations ~~and must be used accordingly.~~

Table 9-1-7. ~~USGS~~ USA Site Factor, F_{pga}

~~USGS Site Class~~	~~Peak Ground Acceleration Coefficient (PGA)~~[1]				
	~~PGA ≤ 0.10~~	~~PGA = 0.20~~	~~PGA = 0.30~~	~~PGA = 0.40~~	~~PGA ≥ 0.50~~
~~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~~[2]	*	*	*	*	*

~~Notes:~~
~~[1]Use straight-line interpolation for intermediate values of PGA.~~
~~[2]Site-specific hazard analysis should be performed for all sites in Site Class F.~~

~~Table 9-1-8.~~ USA ~~USGS Site Factor, F_a~~

~~USGS Site Class~~	~~Spectral Acceleration Coefficient at 0.2 second period (S_s)~~[1]				
	~~$S_s \leq$ 0.25~~	~~$S_s =$ 0.50~~	~~$S_s =$ 0.75~~	~~$S_s =$ 1.00~~	~~$S_s \geq$ 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~~	~~1.2~~	~~1.1~~	~~1.0~~
~~E~~	~~2.5~~	~~1.7~~	~~1.2~~	~~0.9~~	~~0.9~~
~~F~~[2]	*	*	*	*	*

~~Notes:~~
~~[1]Use straight-line interpolation for intermediate values of S_s.~~
~~[2]Site-specific hazard analysis should be performed for all sites in Site Class F.~~

~~Table 9-1-9.~~ USA ~~USGS Site Factor, F_v~~

~~USGS Site Class~~	~~Spectral Acceleration Coefficient at 1.0 second period (S_1)~~[1]				
	~~$S_1 \leq$ 0.10~~	~~$S_1 =$ 0.20~~	~~$S_1 =$ 0.30~~	~~$S_1 =$ 0.40~~	~~$S_1 \geq$ 0.50~~
~~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~~[2]	*	*	*	*	*

~~Notes:~~
~~[1]Use straight-line interpolation for intermediate values of S_1.~~
~~[2]Site-specific hazard analysis should be performed for all sites in Site Class F.~~

Table 9-1-10. ~~GSC~~ Canadian Site Factor, F_a

GSC Site Class	Spectral Acceleration Coefficient at 0.2 second period (S_s)[1]				
	$S_s \leq$ 0.25	$S_s =$ 0.50	$S_s =$ 0.75	$S_s =$ 1.00	$S_s \geq$ 1.25
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F[2]	*	*	*	*	*

Notes:
[1]Use straight-line interpolation for intermediate values of S_s.
[2]Site-specific hazard analysis should be performed for all sites in Site Class F.

Table 9-1-11. Canadian ~~GSC~~ Site Factor, F_v

GSC Site Class	Spectral Acceleration Coefficient at 1.0 second period (S_1)[1]				
	$S_1 \leq$ 0.10	$S_1 =$ 0.20	$S_1 =$ 0.30	$S_1 =$ 0.40	$S_1 \geq$ 0.50
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F[2]	*	*	*	*	*

Notes:
[1]Use straight-line interpolation for intermediate values of S_1.
[2]Site-specific hazard analysis should be performed for all sites in Site Class F.

C - 1.4.4.1 Site Effects

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

~~C - 1.4.4.1.1 Site Class~~

The site classes in this chapter are consistent with those in Reference 4 and ~~and~~ Reference 19. Previous versions of this chapter required site class evaluation that was consistent with older versions of the references; namely, 1) site classes were grouped as A, B, C, D, E, and F, and 2) site class could be evaluated directly from SPT blow counts, soil shear strength, or lab-based soil parameters over the upper 100 feet (30 m) of the soil profile. Consistent with the current versions of Reference 4 and Reference 19, this chapter now requires evaluation of shear wave velocity profile using direct measurements of shear wave velocity or correlations with ~~The stiffness of a site may be classified by the average shear wave velocity, average Standard Penetration Test (~~SPT or CPT measurements. ~~) blow counts or average undrained shear strength of soils in the upper 100 feet (30 m) of the soil profile. Several m~~Methods to assist practitioners in determininge ~~these average values~~ the site classification are presented in Reference 194. ~~, along with steps that may be followed to classify a site. A~~Although direct measurement of shear wave velocity is typically cost effective for large projects or in tandem with CPT testing, practitioners will likely rely on correlations to evaluate shear wave velocity for routine projects that incorporate SPTs. The appropriate correlations for a given project may be based on site-specific relationships or published equations. Numerous published equations are available and state transportation agencies or other public agencies in high-seismicity areas frequently offer reliable relationships applicable to local practice. For a synthesis of many available correlations, please consult [new reference Wair et al, PEER 2012/08].

Do not assume a default site classification without reviewing mapped subsurface conditions at the site ~~is not given, as this would require a judgment based on little to no knowledge of the soils~~. Where a site classification must be assumed, Reference 19 recommends the most critical site conditions and ground motion parameters resulting from Site Class C, Site Class CD, and Site Class D be used for design. This default site class may be unconservative for soft soil conditions corresponding to Site Class E or Site Class F.

Experience has shown that most railroad bridge failures that have occurred in seismic events were due to soil failures such as lateral spreading or liquefaction. Because of this, it is recommended that the foundation investigation should include ~~soil borings or test pits taken~~a subsurface exploration program performed to an adequate depth to ~~determine the soil profile~~evaluate the potential for liquefaction-induced ground failure. It should be emphasized that an adequate foundation investigation is necessary to determine the appropriate foundation type for the structure.

~~C - 1.4.4.1.2 Site Factors~~

~~The site factors are consistent with those in Reference 4 and Reference 19 and Reference 20. Site Class B is the reference site class for the USGS acceleration coefficient maps, and is therefore the site condition for which the USGS site factor is 1.0. Site Class C is the reference site class for the GSC acceleration coefficient maps, and is therefore the site class condition for which the GSC site factor is 1.0. Other Site Classes have separate sets of site factors which generally increase as the soil profile becomes softer. Except for USGS Site Class A and GSC Site Classes A and B, the factors also decrease as the ground motion level increases, due to the strongly nonlinear behavior of the soil.~~

~~Caution should be exercised when applying site factors if acceleration maps other than those discussed in Article 1.3.2.3 are used. In this case it would be appropriate to use site factors consistent with the maps being used.~~

1.4.4.2 Damping Adjustment Factor

The Damping Adjustment Factor, D, may be calculated from the following formula. In the absence of more definitive information, a damping adjustment factor of 1.0 shall be used.

$$D = \left(\frac{1.5}{(0.4\,\xi + 1)} + 0.5\right)$$

D= Damping Adjustment Factor

ξ= Percent Critical Damping (e.g. 5%)

C - 1.4.4.2 Damping Adjustment Factor

The Damping Adjustment Factor provides a simplistic method for scaling the seismic response coefficient to account for different structure types and conditions. The seismic response coefficient is given for 5% critical damping without the damping adjustment factor. The percent critical damping varies based on the structure material and system, effect of structure attachments (i.e., track and ballast), whether the structure responds in the elastic-linear or post-yield range, ~~and~~ whether or not the structure response is dominated by the foundation or abutment response, seismic isolation of the structure, damping systems incorporated into the structure, soil conditions and proximity to faults.

The percent critical damping (ξ) preferably should be based on actual test data from similar structure types, soil conditions, soil-structure interaction analysis, the effects of near-fault or far-fault sites and test data for seismic isolation and damping systems. ~~A table of damping values for different structural (building) systems from Reference 11 is included below for information and guidance.~~

~~*Table 9-C-1. Damping Values for Structural Systems*~~

~~Structural System~~	~~Elastic-Linear~~	~~Post-Yield~~
~~Structural Steel~~	~~3%~~	~~7%~~
~~Reinforced Concrete~~	~~5%~~	~~10%~~
~~Masonry Shear Walls~~	~~7%~~	~~12%~~
~~Wood~~	~~10%~~	~~15%~~
~~Dual Systems~~	~~See note 1~~	~~See note 2~~

~~Notes:~~

~~1. Use the value of the primary, or more rigid, system. If both systems are participating significantly, a weighted value, proportionate to the relative participation of each system, may be used.~~

~~2. The value for the system with the higher damping value may be used.~~

1.4.4.3 Seismic Response Coefficient

The Seismic Response Coefficient, C_m, to be used in the methods of analysis recommended in ~~Article 1.4.5~~ Article 1.4.5, shall be ~~calculated from the following formula~~determined via the multi-period response spectrum developed per Article 1.3.2.3 and accounting for site effects described in Article 1.4.4.~~.~~

For ~~areas~~ sites with soft soil conditions ~~and high seismicity~~, potential seismic-induced ground failure, or close proximity to known faults, use of a site-specific response spectrum is preferred.

$$\cancel{C_m = \frac{F_v S_1 D}{T_m} \le F_a S_S D}$$

$$S_a = f(T)$$

$$C_m = S_a * D$$

~~C_m= Seismic Response Coefficient for the m^{th} mode~~

~~T_m= Period of vibration of the m^{th} mode in seconds~~

S_a= Spectral Response Coefficient determined in accordance with Article 1.3.2.3

$T_{\cancel{m}}$= Period of vibration ~~of the m^{th} mode in seconds~~

C_m= Seismic Response Coefficient for the m^{th} mode

D= Damping Adjustment Factor determined in accordance with ~~Article 1.4.4.2~~Article 1.4.4.2

~~S_S= Short-Period (0.2 second) Spectral Response Accleration Coefficient determined in accordance with Article 1.3.2.3~~

~~S_1= Long-Period (1.0 second) Spectral Response Accleration Coefficient determined in accordance with Article 1.3.2.3~~

~~F_a= Site Factor for short-period range of acceleration spectrum determined in accordance with Article 1.4.4.1~~

~~F_v= Site Factor for long-period range of acceleration spectrum determined in accordance with Article 1.4.4.1~~

~~D= Damping Adjustment Factor determined in accordance with Article 1.4.4.2~~

~~T_m= Period of vibration of the m^{th} mode in seconds~~

C - 1.4.4.3 Seismic Response Coefficient

The Seismic Response Coefficient is the basis for determining the structure design loads for both the Equivalent Lateral Force Procedure and the Modal Analysis Procedure. The Equivalent Lateral Force Procedure only uses a single value based on the natural period of vibration of the structure for each of the two principal directions of the structure. The Modal Analysis Procedure combines values for multiple modes of vibration in each of the two principal directions of the structure.

For areas with soft soil conditions and high seismicity, or close proximity to known faults, or for special bridge projects, a site-specific hazard analysis is preferred. The analysis should be based on accepted practice using the ground motion return period determined in accordance with Article 1.3.2.2 “Structure Importance Classification.” A good discussion of site-specific hazard analysis is contained in Reference 4~~ and Reference 11~~.

The formula for the Seismic Response Coefficient is adopted from Reference 4, rearranged to more closely resemble previous editions of this chapter~~ and modified by the Damping Adjustment Factor from Reference 11~~. ~~The coefficient is based on 5% critical damping. There are exceptions to the formula; however, they were not included since the exceptions differ from code to code and unnecessarily complicate the Seismic Response Coefficient. The values obtained using the basic formula are conservative compared to the exceptions. The exceptions from various codes are listed below for information. If the bridge designer believes that the exceptions are needed for a particular site, they may be included or preferably use site-specific response spectra.~~

~~Table 9-C-2. Exceptions to Seismic Response Coefficient~~

~~Source~~	~~Exception~~
~~Reference 2~~	~~For long period bridges (greater than about 3 seconds) response accelerations are proportional to 1/T²~~.
~~Reference 12~~	~~For structures where any modal period of vibration (T_m) exceeds T_1, the long period transition period (varies between 4.0 seconds and 16.0 seconds), the Seismic Response Coefficient for that mode is permitted to be determined by the following equation:~~ $C_m = \frac{F_v S_1 D T_L}{T_m^2}$

1.4.4.4 Structural Flexibility for Low Period ~~Reduced~~ Response

~~The~~ When the structure period, T, is on the ascending branch of the response spectrum then additional flexibility in the structure will increase seismic demands. Conservatively the maximum spectral acceleration can be used, otherwise all potential unaccounted sources of flexibility shall be considered. Common additional sources of flexibility include the following~~seismic response of the bridge may be reduced in accordance with Paragraph 1.4.4.4b if the following provisions are satisfied.~~:

(1) ~~The period, T, of the bridge is~~ Stiffness of reinforced concrete substructure members determined using the effective moment of inertia, I_e~~, for reinforced concrete substructure members. The effective moment of inertia may be calculated using EQ 2-12 in Chapter 8, Part 2, Paragraph 2.23.7c.~~

(2) ~~The period, T, of the bridge is determined including the effects of f~~Foundation flexibility effects~~.~~

(3) ~~The bridge response considers the l~~Lateral flexibility of the spans between piers.

(4) Foundation rocking effects~~The effects of foundation rocking are accounted for if the moment due to seismic loads exceeds the overturning moment of the footing.~~

~~The seismic response coefficient, C_m, for bridge structures with periods less than the initial transition period, T_o, may be determined as follows:~~

$$C_m = F_{pga} PGA \text{ for } T_m \leq 0.03 \text{ seconds}$$

$$C_m = F_{pga} PGA + \frac{(T_m - 0.03)(F_a S_S D - F_{pga} PGA)}{(T_o - 0.03)} \text{ for } 0.03 < T_m \leq T_o \text{ seconds}$$

~~C_m = Seismic Response Coefficient for the m^{th} mode~~
~~PGA = Peak Ground Acceleration Coefficient determined in accordance with Article 1.3.2.3~~
~~F_{PGA} = Site Factor for peak ground acceleration determined in accordance with Article 1.4.4.1 (for GSC based acceleration F_{pga} shall be replaced with F_a)~~
~~D = Damping Adjustment Factor determined in accordance with Article 1.4.4.2~~
~~T_o = Initial transition period = $0.2(F_v S_1/F_a S_S)$ in seconds~~
~~T_m = Period of vibration of the m^{th} mode in seconds~~

C - 1.4.4.4 Structural Flexibility for Low Period ~~Reduced~~ Response

Railroad bridges are often more rigid than typical multi-level buildings or highway bridge structures. ~~Therefore~~Therefore, the response of railroad bridges in the low period range needs to be thoroughly addressed. Underestimation of the structure period can result in unconservative response for low period structures when the reduced response region of the response spectra is used. ~~This section was developed to allow the bridge designer to take advantage of the reduced response for low period structures when appropriate.~~ The provisions listed in Article 1.4.4.4 account for the most common sources of flexibility in the structure, however, the bridge designer should consider any other component that will increase the structure period.

~~Most general response spectra curves, such as those defined in Reference 13 have reduced responses in the low period range. Typically, these curves vary linearly from the peak ground acceleration at zero period to a maximum constant acceleration response at the initial transition period, T_o as shown in Figure 9-C-2. Other response spectra curves, such as those given in Reference 5 show a flat region for very low periods that represent perfectly rigid response. The AREMA seismic response coefficient defined in Article 1.4.4.3 does not include the reduced response at low periods since it was felt that typical railroad bridge analysis underestimates the actual period of the bridge. Underestimation of the structure period can result in unconservative response for low period structures when the reduced response region of the response spectra is used. This section was developed to allow the bridge designer to take advantage of the reduced response for low period structures when appropriate. The provisions listed in Article 1.4.4.4 account for the most common sources of flexibility in the structure, however, the bridge designer should consider any other component that will increase the structure period.~~

Typical railroad bridge analysis uses the gross moment of inertia for reinforced concrete members to determine the stiffness and load distribution. Use of the gross moment of inertia for a reinforced concrete substructure member will underestimate the structure period when the flexural tension stress exceeds the concrete modulus of rupture. The effective moment of inertia, as determined from EQ 2-12 in Chapter 8, Part 2, Article 2.23.7c, of reinforced concrete members will provide a more representative structure period. The cracked moment of inertia used in EQ 2-12 may be determined from moment-curvature analysis of the member using the following relationship.

$$I_{cr} = \frac{M_{y1}}{E_c \phi_{y1}}$$

M_{y1} = Moment at first yield of reinforcing steel
ϕ_{y1} = Curvature at first yield of reinforcing steel
E_c = Concrete modulus of elasticity (Chapter 8, Part 2, Article 2.23.4)

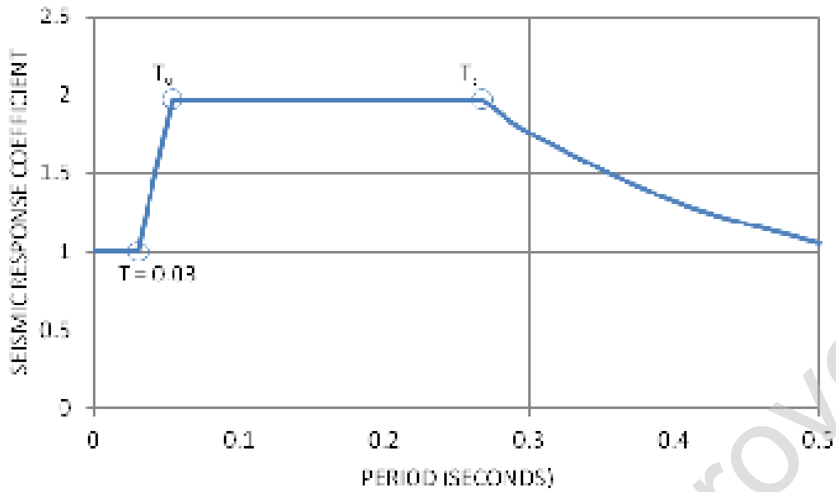
It is common practice to model bridge foundations as either pinned or fixed. If the foundation stiffness is overestimated, then the structure period will be underestimated. Foundation flexibility for spread footings may be accounted for by including a rotational footing stiffness calculated in accordance with accepted procedures, such as those defined in Section 5.3 of Reference 17. Lateral translation flexibility of a spread footing need not be considered provided that the base soil friction is not exceeded. Foundation flexibility for pile footings may be accounted for by using accepted procedures, such as including a rotational pile cap stiffness that is derived from realistic pile load-deflection (t-z) data. When vertical piles are used, the lateral translation foundation stiffness should be determined from realistic pile lateral load-deflection (p-y) data, supplemented, if appropriate, by lateral soil resistance on the pile cap. If either of these foundation types is founded on sound rock, the effects of foundation flexibility can be neglected.

Lateral flexibility of the bridge spans may amplify the seismic response between the bridge piers. For example, a point in the middle of the span may have a higher response acceleration than the point at the top of the pier. This effect is typically accounted for by performing modal analysis ~~of the bridge using a model with at least four elements making up the span length~~ on bridge models capturing sufficient lateral degrees of freedom along the span.

Foundation rocking is a response that occurs when the applied moment on a spread footing exceeds the overturning moment resistance. Rocking response will increase the period of the foundation and most likely take it out of the low period reduced response range.

~~The low period reduced response defined in this Article has been developed based on review of the response spectra from other codes along with visual inspection of a number of response spectra generated from actual strong motion records. The perfectly-rigid period limit of 0.03 seconds corresponds to a frequency of 33 Hz and has generally been considered appropriate for this type of response. Evaluation of response spectra generated from actual strong motion records indicates that this is conservative except for sites very close (< 10 miles or 16 km) to the fault. The only structures that are expected to fall in the perfectly-rigid range are rigid piers with spread footings or piles founded on rock. Other rigid piers will generally fall in the low period linear transition region due to foundation flexibility.~~

Note that response spectra curves in previous editions of AREMA showed a conservative flat region in the low period range. The accompanying commentary allowed for a low period reduced response spectral shape adjustment only if potential unaccounted sources of structure flexibility are considered. Given advancements in seismic hazard calculations as well as overall railroad bridge analysis practice it was felt that conservative flat region for low periods was no longer necessary. Rather, this provision provides caution for bridges in low period range and requires consideration of all potential sources of structure flexibility. This approach allows for better alignment with seismic hazard products being provided from sources such as USGS and GSC.



$0.00 < T \leq 0.03$ Perfectly-rigid region

$0.03 < T \leq T_o$ Low period linear transition region

$T_o < T \leq T_s$ Constant acceleration region

T_o = Initial transition period = $0.2(F_vS_1/F_aS_s)$

T_s = Constant acceleration transition period = F_vS_1/F_aS_s

T = Period of vibration

S_S = Short-period (0.2 second) Spectral Response Acceleration Coefficient determined in accordance with Article 1.3.2.3

S_1 = Long-period (1.0 second) Spectral Response Acceleration Coefficient determined in accordance with Article 1.3.2.3

F_a = Site Factor for short-period range of acceleration spectrum determined in accordance with Article 1.4.4.1

F_v = Site Factor for long-period range of acceleration spectrum determined in accordance with Article 1.4.4.1

Figure 9-C-2. Example Response Spectra with Low Period Reduced Response

1.4.5.3 Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure may be used for two-span bridges or multi-span regular bridges as described in Article 1.4.5.2. The procedure is described below.

a. Calculate the Seismic Response Coefficient (C_m) for each of the two principal directions of the structure as follows.

   (1) Calculate the natural period of vibration (T_m) for each of the two principal directions of the structure using any commonly accepted method.

   (2) Calculate the Seismic Response Coefficient (C_m) for each of the two principal directions of the structure from Seismic Response Coefficient “Seismic Response Coefficient.”

b. Perform static analysis on the bridge in each of the two principal directions.

   (1) Calculate the distributed seismic load in each direction from the following formula.

   $p(x)$ = distributed seismic load per unit length of bridge

   C_m = Seismic Response Coefficient

   $w(x)$ = distributed weight of bridge per unit length

$$p(x) = C_m w(x)$$

   (2) Distribute the seismic load to individual members based on the stiffness and support conditions.

c. Combine the loads in each of the two principal directions of the structure to get the final seismic design loads.

   (1) Combination 1: Combine the forces in principal direction 1 with 30% of the forces from principal direction 2.

   (2) Combination 2: Combine the forces in principal direction 2 with 30% of the forces from principal direction 1.

C - 1.4.5.3 Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure is included as a simple method of analysis that may be used for regular bridges. The calculations for this procedure are appropriate for hand calculation methods in most cases, though static computer analysis may be used to determine the load distribution to the individual members.

The two principal directions of the structure are typically the longitudinal and transverse directions of the bridge. For curved bridges, the longitudinal direction may be taken as a straight line connecting the centerline of the bridge at the beginning and end.

The natural period of vibration (T_m) for each of the two principal directions of the structure may be calculated using any commonly accepted method. The following simple formulation may be used.

$$T_m = 2\pi\sqrt{\frac{W}{gK}}$$

W= Total weight of the bridge.

g= Acceleration due to gravity (length/time^2)

K= The total structure stiffness including the stiffness of the superstructure, supporting members and surrounding soil.

The actual seismic response coefficient, C_m, varies throughout the structure in proportion to the relative lateral movement. A common method of equivalent lateral force analysis assumes that one-half the weight of the substructure is lumped at the superstructure level for the period calculation and the foundation load is calculated using the complete bridge weight with the seismic response coefficient determined for the superstructure. This analysis approach is accurate when the substructure weight is small relative to the superstructure weight, but may be too conservative for heavy pier substructures. Rather than using the more rigorous modal analysis approach, a simple modification to the equivalent lateral force procedure may be used to determine a less conservative foundation demand for bridges supported by heavy pier substructures. For single level bridges, it is conservative to assume that the actual seismic response coefficient, C_m, varies linearly from the peak ground acceleration (PGA) response coefficient at the ground level to the seismic response coefficient calculated at the superstructure level. Therefore, ~~The seismic response coefficient, C_m, applied to the substructure of single level bridges~~application of C_m on single level bridge substructures ~~-~~may be ~~reduced~~ simplified ~~to the~~by taking the average of the C_m value calculated in Paragraph 1.4.5.3a for the superstructure and the ~~peak ground acceleration~~ PGA response coefficient ~~multiplied by the appropriate site factor, F_{pga}PGA,~~ determined in accordance with Article 1.3.2.3 for the ground~~,~~. However, this average C_m response ~~but~~ shall ~~not be~~never be taken as less than the ~~peak ground acceleration~~PGA response coefficient~~, multiplied by the appropriate site factor, F_{pga}PGA~~. ~~The actual seismic response coefficient, C_m, varies throughout the structure in proportion to the relative lateral movement.~~ ~~A common method of equivalent lateral force analysis assumes that one-half the weight of the substructure is lumped at the superstructure level for the period calculation and the foundation load is calculated using the complete bridge weight with the seismic response coefficient determined for the superstructure. This analysis approach is accurate when~~

~~the substructure weight is small relative to the superstructure weight, but may be too conservative for heavy pier substructures. Rather than using the more accurate modal analysis approach, a simple modification to the equivalent lateral force procedure may be used to minimize the foundation demand for bridges supported by large pier substructures. It is conservative to assume that the actual seismic response coefficient,~~ ~~C_m~~~~, varies linearly from the peak ground acceleration coefficient, multiplied by the appropriate site factor,~~ ~~F_{pga}~~~~PGA, at the ground level to the seismic response coefficient calculated at the superstructure level as long as the response at the superstructure level exceeds the peak ground acceleration coefficient multiplied by the appropriate site factor. Therefore the average of these two acceleration values may be applied to the weight of the pier to more accurately determine the demand at the foundation.~~

The seismic load should be distributed to the individual members based on the stiffness and support conditions. For a regular structure with uniform weight per unit length and simple supports, this reduces to a simple beam calculation for the superstructure between supports and a single lateral load calculation for the supporting bents.

1.4.5.4 Modal Analysis Procedure

The Modal Analysis Procedure may be used for any structure configuration except complex bridge configurations as described in Article 1.4.5.2. The procedure is described below.

a. Develop ~~elastic~~ the response ~~spectra~~ spectrum from Seismic Response Coefficient "Seismic Response Coefficient."

b. Perform dynamic analysis on the structure in each of the two principal directions using the ~~elastic~~ response ~~spectra~~ spectrum to determine the individual member loads.

   (1) A mathematical model should be used to calculate the mode shapes, frequencies and member forces. The model should accurately represent the structure mass, stiffness and support conditions.

   (2) The structural responses should be calculated from an appropriate modal combination technique

   ~~(2)~~(3) An adequate number of modes should be included so that the response in each principal direction includes a minimum 90% mass participation.

c. Combine the loads in each of the two principal directions of the structure using one of the following methods to get the final seismic design loads.

   (1) SRSS Method - Combine forces in individual members using the square root of the sum of the squares from each principal direction.

   (2) Alternate Method - Perform two load combinations for investigation.

      (a) Combination 1: Combine the forces in principal direction 1 with 30% of the forces from principal direction 2.

      (b) Combination 2: Combine the forces in principal direction 2 with 30% of the forces from principal direction 1.

C - 1.4.5.4 Modal Analysis Procedure

The Modal Analysis Procedure is included as a general method of analysis that may be used for any bridge configuration except complex configurations. The calculations for this procedure are appropriate to be performed by any commonly available finite element computer program.

~~The~~ R~~r~~esponse spectra ~~is~~ used in the modal analysis procedure~~developed from Paragraph 1.4.4.3 "Seismic Response Coefficient." The value of the Seismic Response Coefficient (C_m)~~ should ~~be calculated~~ have a well-defined spectral shape ~~for a range of period (T_m) values to~~over the structure response period range ~~adequately define the spectral shape for the range of period (T_m) values needed to represent the~~of interest ~~structure.~~_ ~~Figure 9-C-3 gives an example spectral shape for values of F_v, F_1, S_1 and D all equal to 1.0 and S_S equal to 2.5.~~

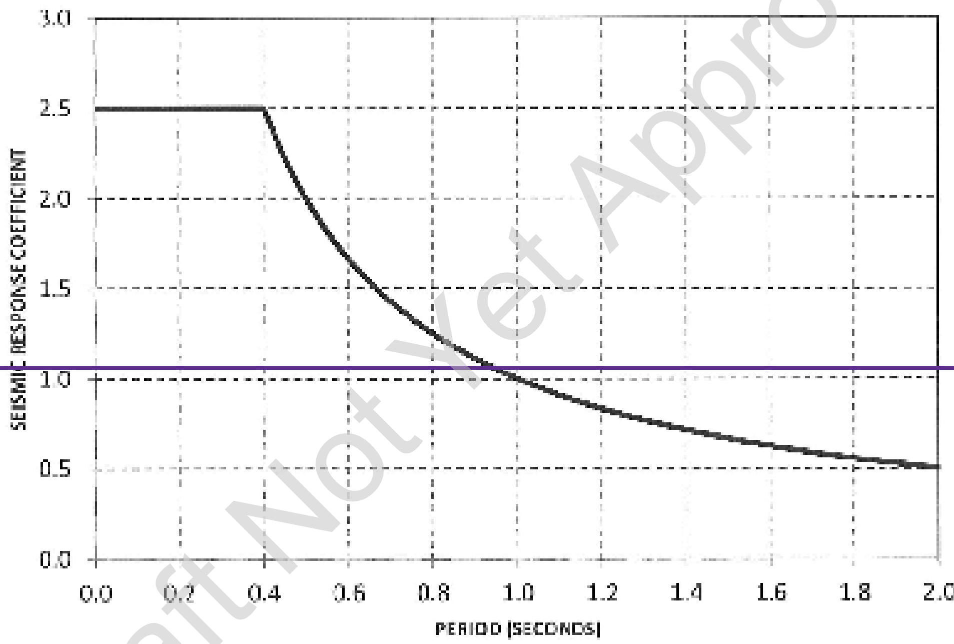


Figure 9-C-3. Example Response Spectra

References

— ~~2014~~ 2024 —

Updated References

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~~FEMA (1997a), “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, FEMA 273 Report, Federal Emergency Management Agency, Washington, D.C.~~

~~FEMA (1997b), “NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings”, FEMA 274 Report, Federal Emergency Management Agency, Washington, D.C.~~

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Incorporation of Proposed Revisions (Track Changes Accepted)

1.3.2.3 Acceleration Coefficient Maps

Acceleration coefficient maps are provided in this Article to help define the seismic hazard. Figures 9-1-1 through 9-1-3 show peak ground accelerations in the United States for return periods of 100 years, 475 years and 2475 years. These maps are mainly for illustration purposes and more accurate acceleration coefficients may be determined using web-based interactive tools found on the United States Geological Survey (USGS) website. Acceleration coefficients for sites located in Canada may be determined using the tools found on the Natural Resources Canada (NRC) website. Other sources or site-specific procedures may be used to define the base accelerations as long as they are based on accepted methods.

The USGS tools allow direct determination of acceleration for any return period. The NRC tools provide accelerations for 10 discrete probabilities of exceedance. The NRC probabilities of exceedance correspond to return periods shown in Table 9-1-6. Accelerations for return periods other than those shown may be determined from log-log (base 10) interpolation/extrapolation.

Table 9-1-6. Return Periods for NRC Probabilities of Exceedance

Probability of exceedance in 50 years	Return period in years
2%	2475
3%	1642
4%	1225
5%	975
7%	689
10%	475
14%	332
20%	225
30%	141
40%	98

C - 1.3.2.3 Acceleration Coefficient Maps

Acceleration coefficient maps reflect the seismic hazard at a site. They account for both maximum ground motion intensity expected and frequency of occurrence. The maps give acceleration levels with a uniform probability of being exceeded in all areas of the country. The steps involved in the development of these maps include: (1) the definition of the nature and location of earthquake sources, (2) magnitude-frequency relationships for the source, (3) attenuation of ground motion with distance from the source, and (4) determination of ground motion parameters at the site having the required probability of exceedance.

The peak ground acceleration maps for return periods of 100 years, 475 years and 2475 years in the United States were prepared by the United States Geological Society (USGS) for AREMA. These maps are included mainly for illustrative purposes. Procedures for determining design accelerations for sites located in the United States and Canada are described in the following paragraphs.

Accelerations for sites in the United States may be determined by using the interactive tools found on the USGS website at https://earthquake.usgs.gov/nshmp/. Determination of accelerations for sites in Canada will require the use of a web-based hazard calculator found on the Natural Resources Canada (NRC) website at https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php. Example procedures for each website are shown below. The acceleration values shown are for example purposes only and should not be used for design.

Procedure for sites in the United States

b. Navigate to https://earthquake.usgs.gov/nshmp/. Select the 'Hazard Curves (static)' link. A page will appear that requires the user to select a Model, Location, Site Class and Return Period.

c. For this example, select 'Static Hazard Curves for the 2018 Conterminous U.S.' from the dropdown menu for the Model.

d. Type in site location information (for this example Latitude = 33.06277, Longitude = -115.759).

e. Select Site Class (for this example use BC).

f. Enter return period (for this example a return period = 100 years was used).

g. Select 'Plot'. Hazard Curves and a Uniform Hazard Response Spectrum will appear. Acceleration values may be determined by selecting the 'Response Spectrum Data' tab. Acceleration values for this example are as follows:

PGA	40% in 50 yrs	0.2766 (g)
PGA	10% in 50 yrs	0.5408 (g)
PGA	2% in 50 yrs	0.8976 (g)

Lat: 33.06277, Lon: -115.759

Notes:

40% in 50 yrs = ~100-year Return Period

10% in 50 yrs = 475-year Return Period

2% in 50 yrs = 2475-year Return Period

Procedure for sites in Canada

a. Navigate to https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php

b. Locate and select '2020 National Building Code of Canada Seismic Hazard Tool'

c. Enter the shear wave velocity (V_{s30}) or select the site class (X_s). For this example, select site class C.

d. Enter the Latitude and Longitude of the site under consideration (for this example Latitude = 48.4133, Longitude = -71.0666) and select Set coordinates.

e. Click on 'Obtain Seismic Hazard Values'

The page will reload after the calculation is complete. Scroll down to the acceleration values which will appear similar to this:

2%/50 years (0.000404 per annum) probability

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
1.11 g	0.656 g	0.347 g	0.155 g	0.592 g

5%/50 years (0.001 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.645 g	0.37 g	0.187 g	0.081 g	0.351 g

10%/50 years (0.0021 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.401 g	0.229 g	0.112 g	0.0472 g	0.145 g

Additional return periods are available by selecting the "Additional Values" tab and selecting a probability value (% exceedance in 50 years) from the drop-down menu. For this example, 40% was selected representing a 100 year return period.

40%/50 years (0.01 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA
0.119 g	0.0664 g	0.0301 g	0.0117 g	0.0636 g

Notes:

2%/50 years = 2475-year Return Period

5%/50 years = 975-year Return Period

10%/50 years = 475-year Return Period

40%/50 years = ~100-year Return Period

Future earthquakes and earthquake research will continue to improve the overall understanding of the seismic hazard and will result in revisions to the acceleration maps. The 2018 edition of the USGS maps and the 2020 edition of the NRC maps were used in the examples above. More recent maps, maps from different sources, or site-specific procedures may be used as long as they are based on accepted methods and are consistent with the site conditions and response spectra equations in Article 1.4.4.

Accelerations for return periods other than those shown on the NRC maps may be estimated using log-log (base 10) interpolation/extrapolation between listed return periods. This approach is based on the procedure shown in Article A-4.1.8.4(6) of Reference 19.

For example, using the values for site location at Latitude = 48.4133, Longitude = -71.0666 and return period of 400 years, the peak ground acceleration value is determined as shown below.

PGA(475) = 0.222 from NRC website

PGA(332) = 0.173 from NRC website

Log(PGA(400)) = log(0.173) + (log(0.222)-log(0.173)) x (log(400)-log(332))/(log(475)-log(332))

Log(PGA(400))=-0.7056

PGA(400) =10^0.7056 = 0.197

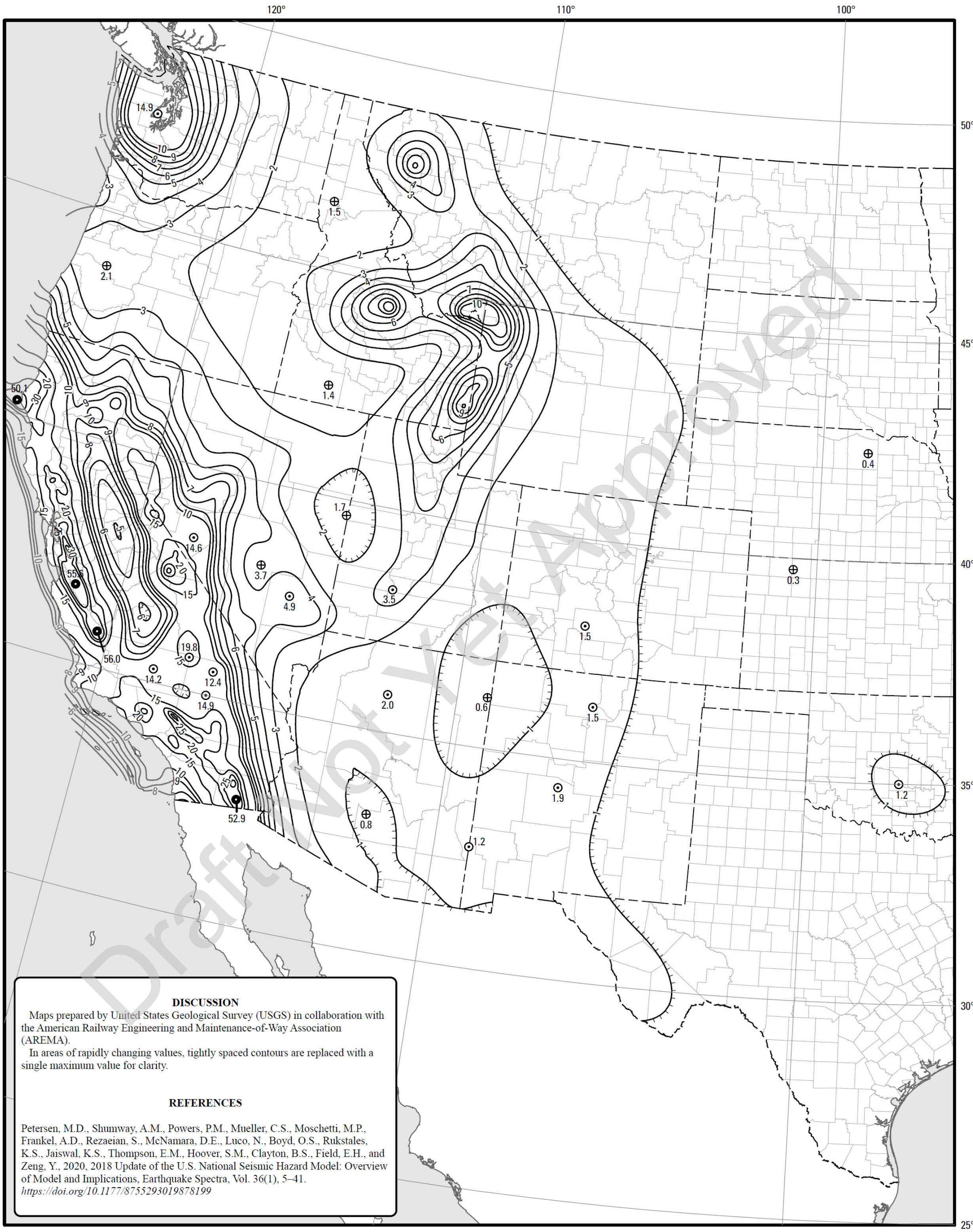

Figure 9-1-1. 100-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States

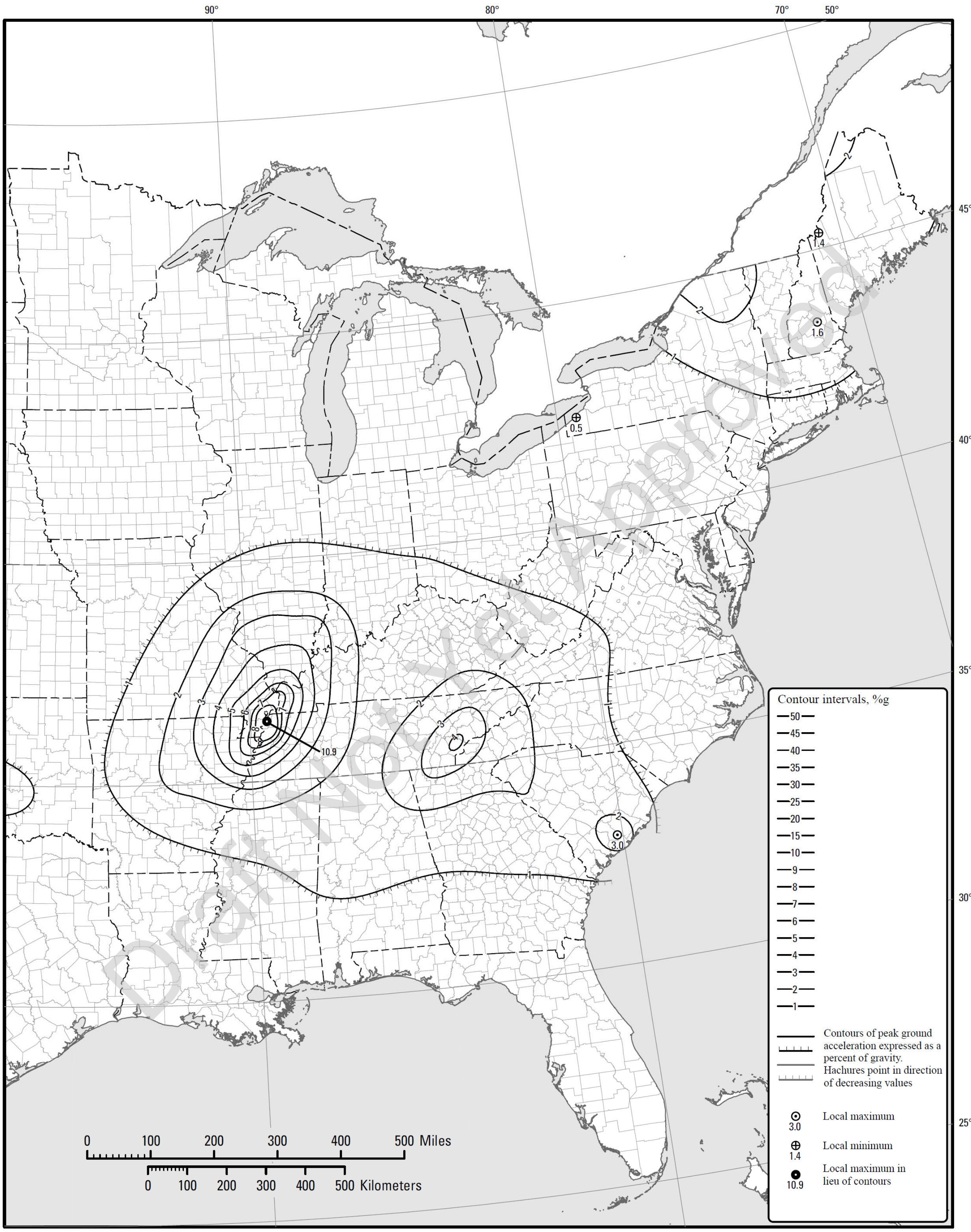

Figure 9-1-1. 100-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

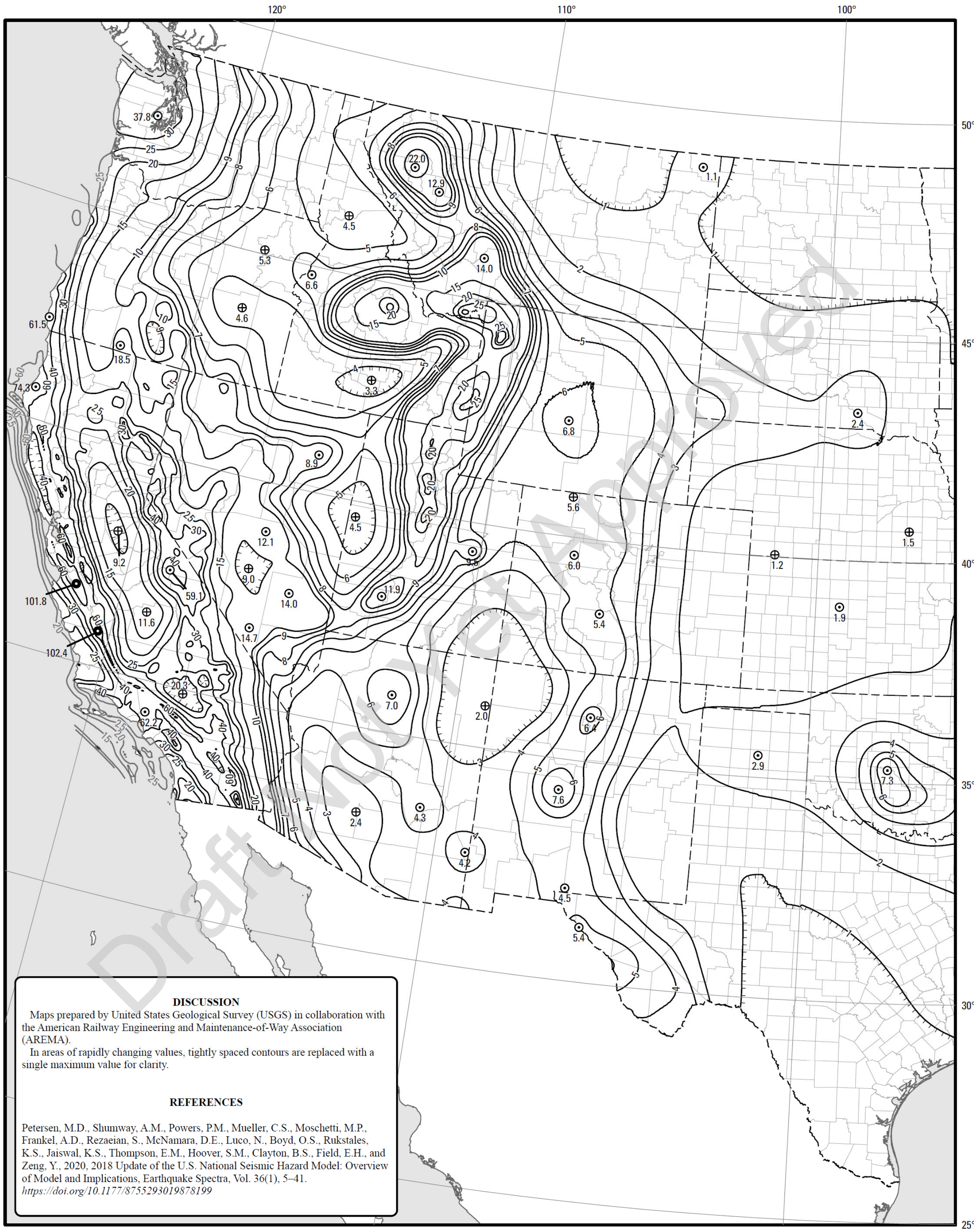

Figure 9-1-2. 475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States

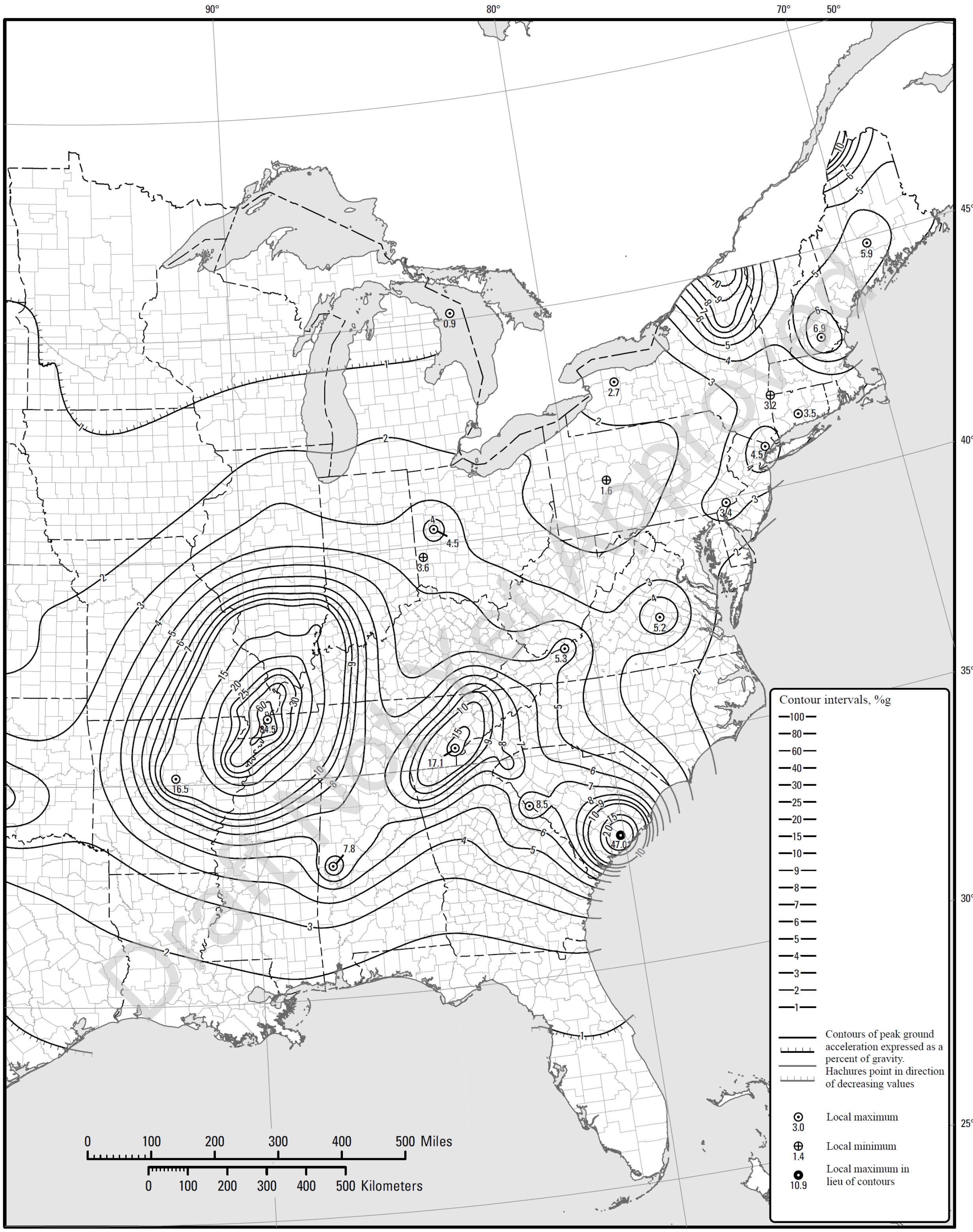

Figure 9-1-2. 475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

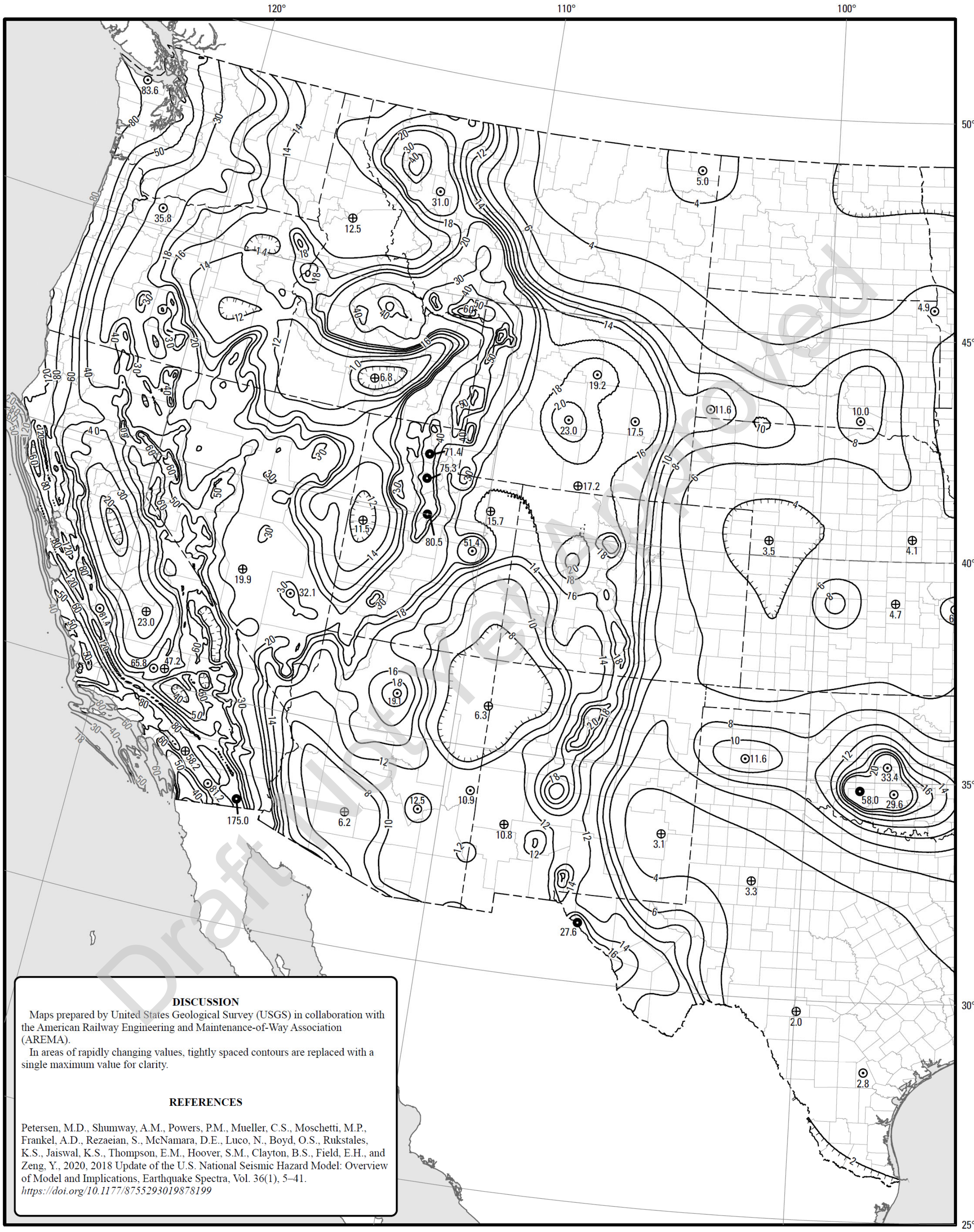

Figure 9-1-3. 2475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration — United States

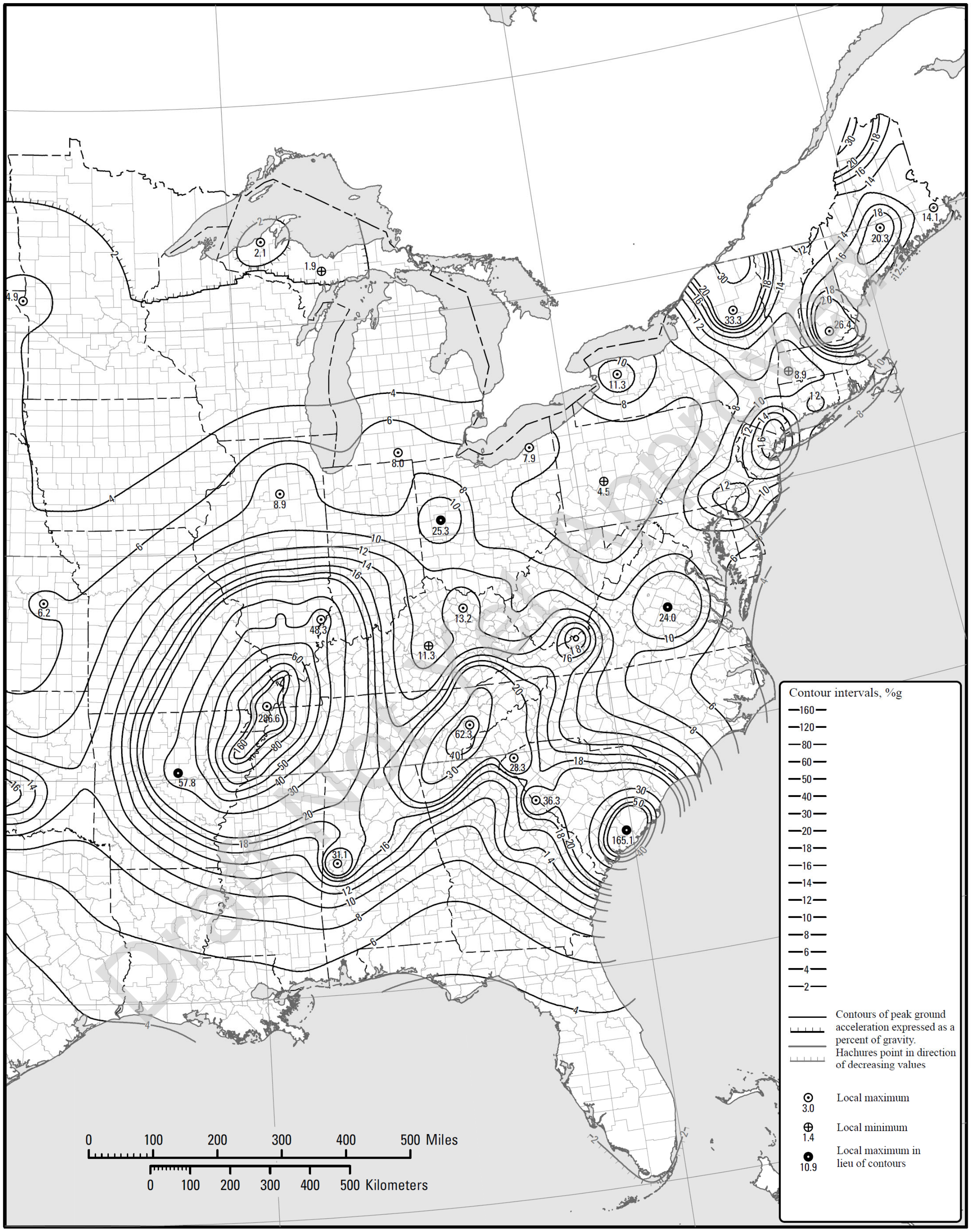

Figure 9-1-3. 2475-year Return Period, Site Class B/C, Horizontal Peak Ground Acceleration – United States (Continued)

1.4.4.1 Site Effects

The effects of site conditions on the response spectrum shall be determined based on the foundation soil characteristics.

A site shall be classified as Site Class A, B, BC, C, CD, D, DE, E, or F in accordance with Site Cl. Sites shall be classified by their time-weighted average shear wave velocity in the upper 100 feet (30 m) of the soil profile, which is defined by the $\bar{v}_s$ parameter. The $\bar{v}_s$ parameter is calculated as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where

$\bar{v}_s$ = the time-weighted average shear wave velocity for the upper 100 feet (30 m) of the soil profile

d_i = the thickness of any soil or rock layer between 0 and 100 feet (30 m);

v_{si} = the shear wave velocity in feet per second (m/s); and

the summation $\sum_{i=1}^{n} d_i$ is equal to 100 feet (30 m).

Table 9-1-6. Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $v_s > 5{,}000$ ft/s (1,524 m/s)
B	Rock with 3,000 ft/s (915 m/s) $< v_s \leq 5{,}000$ ft/s (1,524 m/s)
BC	Rock with 2,100 ft/s (640 m/s) $< v_s \leq 3{,}000$ ft/s (915 m/s)
C	Very dense or hard soil and soft rock with 1,450 ft/s (442 m/s) $< v_s \leq 2{,}100$ ft/s (640 m/s)
CD	Dense or very stiff soil with 1,000 ft/s (305 m/s) $< v_s \leq 1{,}450$ ft/s (442 m/s)
D	Medium dense or stiff soil with 700 ft/s (213 m/s) $\leq v_s \leq 1{,}000$ ft/s (305 m/s)
DE	Loose or medium stiff soil with 500 ft/s (152 m/s) $\leq v_s \leq 700$ ft/s (213 m/s)
E	Very loose or soft soil with $v_s < 500$ ft/s (152 m/s), or with
F	Soils requiring site-specific evaluations, such as: • Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. • Peats or highly organic clays (H > 10 feet (3 m) of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 feet (7.6 m) with PI > 75) • Very thick, soft/medium stiff clays (H > 120 feet (37 m) with $s_u < 1.0$ ksf (48 k Pa)

The $\bar{v}_s$ parameter should be derived from the measured shear wave velocity profile or, if shear wave velocity measurements are not available, from appropriate correlations with standard penetration test (SPT) blow counts, cone penetration test (CPT) resistance measurements, or soil strength and index properties from laboratory testing. Correlations may be based on site-specific relationships or published equations. See the commentary for guidance on selecting appropriate correlations.

If shear wave velocity measurements are not available for the site, the site class should be derived for $\bar{v}_s$, $1.3\bar{v}_s$, and $\bar{v}_s/1.3$, to account for uncertainities associated with estimating the shear wave velocity profile from SPT, CPT, or lab-based correlations. Ground motion parameters should then be developed for design using the most critical of the site classes resulting from $\bar{v}_s$, $1.3\bar{v}_s$, and $\bar{v}_s/1.3$ at each period in the multi-period response spectra.

C - 1.4.4.1 Site Effects

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify ground motions propagated from the underlying rock, sometimes by factors of two or more. The extent of this amplification is dependent on the soil profile at the site and the intensity of shaking in the rock below.

The site classes in this chapter are consistent with those in Reference 4 and Reference 19. Previous versions of this chapter required site class evaluation that was consistent with older versions of the references; namely, 1) site classes were grouped as A, B, C, D, E, and F, and 2) site class could be evaluated directly from SPT blow counts, soil shear strength, or lab-based soil parameters over the upper 100 feet (30 m) of the soil profile. Consistent with the current versions of Reference 4 and Reference 19, this chapter now requires evaluation of shear wave velocity profile using direct measurements of shear wave velocity or correlations with SPT or CPT measurements. Methods to assist practitioners in determining the site classification are presented in Reference 19. Although direct measurement of shear wave velocity is typically cost effective for large projects or in tandem with CPT testing, practitioners will likely rely on correlations to evaluate shear wave velocity for routine projects that incorporate SPTs. The appropriate correlations for a given project may be based on site-specific relationships or published equations. Numerous published equations are available and state transportation agencies or other public agencies in high-seismicity areas frequently offer reliable relationships applicable to local practice. For a synthesis of many available correlations, please consult [new reference Wair et al, PEER 2012/08].

Do not assume a default site classification without reviewing mapped subsurface conditions at the site. Where a site classification must be assumed, Reference 19 recommends the most critical site conditions and ground motion parameters resulting from Site Class C, Site Class CD, and Site Class D be used for design. This default site class may be unconservative for soft soil conditions corresponding to Site Class E or Site Class F.

Experience has shown that most railroad bridge failures that have occurred in seismic events were due to soil failures such as lateral spreading or liquefaction. Because of this, it is recommended that the foundation investigation should include a subsurface exploration program performed to an adequate depth to evaluate the potential for liquefaction-induced ground failure. It should be emphasized that an adequate foundation investigation is necessary to determine the appropriate foundation type for the structure.

1.4.4.2 Damping Adjustment Factor

The Damping Adjustment Factor, D, may be calculated from the following formula. In the absence of more definitive information, a damping adjustment factor of 1.0 shall be used.

$$D = \left(\frac{1.5}{(0.4\,\xi + 1)} + 0.5\right)$$

D= Damping Adjustment Factor

ξ= Percent Critical Damping (e.g. 5%)

C - 1.4.4.2 Damping Adjustment Factor

The Damping Adjustment Factor provides a simplistic method for scaling the seismic response coefficient to account for different structure types and conditions. The seismic response coefficient is given for 5% critical damping without the damping adjustment factor. The percent critical damping varies based on the structure material and system, effect of structure attachments (i.e., track and ballast), whether the structure responds in the elastic-linear or post-yield range, whether or not the structure response is dominated by the foundation or abutment response, seismic isolation of the structure, damping systems incorporated into the structure, soil conditions and proximity to faults.

The percent critical damping (ξ) preferably should be based on actual test data from similar structure types, soil conditions, soil-structure interaction analysis, the effects of near-fault or far-fault sites and test data for seismic isolation and damping systems.

1.4.4.3 Seismic Response Coefficient

The Seismic Response Coefficient, C_m, to be used in the methods of analysis recommended in Article 1.4.5, shall be determined via the multi-period response spectrum developed per Article 1.3.2.3 and accounting for site effects described in Article 1.4.4.

For sites with soft soil conditions, potential seismic-induced ground failure, or close proximity to known faults, use of a site-specific response spectrum is preferred.

$$S_a = f\,(T)$$

$$C_m = S_a * D$$

S_a= Spectral Response Coefficient determined in accordance with Article 1.3.2.3

T= Period of vibration

C_m= Seismic Response Coefficient for the m^{th} mode

D= Damping Adjustment Factor determined in accordance with Article 1.4.4.2

C - 1.4.4.3 Seismic Response Coefficient

The Seismic Response Coefficient is the basis for determining the structure design loads for both the Equivalent Lateral Force Procedure and the Modal Analysis Procedure. The Equivalent Lateral Force Procedure only uses a single value based on the natural period of vibration of the structure for each of the two principal directions of the structure. The Modal Analysis Procedure combines values for multiple modes of vibration in each of the two principal directions of the structure.

For areas with soft soil conditions and high seismicity, or close proximity to known faults, or for special bridge projects, a site-specific hazard analysis is preferred. The analysis should be based on accepted practice using the ground motion return period determined in accordance with Article 1.3.2.2 "Structure Importance Classification." A good discussion of site-specific hazard analysis is contained in Reference 4.

The formula for the Seismic Response Coefficient is adopted from Reference 4, rearranged to more closely resemble previous editions of this chapter.

1.4.4.4 Structural Flexibility for Low Period Response

When the structure period, T, is on the ascending branch of the response spectrum then additional flexibility in the structure will increase seismic demands. Conservatively the maximum spectral acceleration can be used, otherwise all potential unaccounted sources of flexibility shall be considered. Common additional sources of flexibility include the following:

(1) Stiffness of reinforced concrete substructure members determined using the effective moment of inertia, I_e

(2) Foundation flexibility effects

(3) Lateral flexibility of the spans between piers.

(4) Foundation rocking effects

C - 1.4.4.4 Structural Flexibility for Low Period Response

Railroad bridges are often more rigid than typical multi-level buildings or highway bridge structures. Therefore, the response of railroad bridges in the low period range needs to be thoroughly addressed. Underestimation of the structure period can result in unconservative response for low period structures when the reduced response region of the response spectra is used. The provisions listed in Article 1.4.4.4 account for the most common sources of flexibility in the structure, however, the bridge designer should consider any other component that will increase the structure period.

Typical railroad bridge analysis uses the gross moment of inertia for reinforced concrete members to determine the stiffness and load distribution. Use of the gross moment of inertia for a reinforced concrete substructure member will underestimate the structure period when the flexural tension stress exceeds the concrete modulus of rupture. The effective moment of inertia, as determined from EQ 2-12 in Chapter 8, Part 2, Article 2.23.7c, of reinforced concrete members will provide a more representative structure period. The cracked moment of inertia used in EQ 2-12 may be determined from moment-curvature analysis of the member using the following relationship.

$$I_{cr} = \frac{M_{y1}}{E_c \phi_{y1}}$$

M_{y1} = Moment at first yield of reinforcing steel
ϕ_{y1} = Curvature at first yield of reinforcing steel
E_c = Concrete modulus of elasticity (Chapter 8, Part 2, Article 2.23.4)

It is common practice to model bridge foundations as either pinned or fixed. If the foundation stiffness is overestimated, then the structure period will be underestimated. Foundation flexibility for spread footings may be accounted for by including a rotational footing stiffness calculated in accordance with accepted procedures, such as those defined in Section 5.3 of Reference 17. Lateral translation flexibility of a spread footing need not be considered provided that the base soil friction is not exceeded. Foundation flexibility for pile footings may be accounted for by using accepted procedures, such as including a rotational pile cap stiffness that is derived from realistic pile load-deflection (t-z) data. When vertical piles are used, the lateral translation foundation stiffness should be determined from realistic pile lateral load-deflection (p-y) data, supplemented, if appropriate, by lateral soil resistance on the pile cap. If either of these foundation types is founded on sound rock, the effects of foundation flexibility can be neglected.

Lateral flexibility of the bridge spans may amplify the seismic response between the bridge piers. For example, a point in the middle of the span may have a higher response acceleration than the point at the top of the pier. This effect is typically accounted for by performing modal analysis on bridge models capturing sufficient lateral degrees of freedom along the span.

Foundation rocking is a response that occurs when the applied moment on a spread footing exceeds the overturning moment resistance. Rocking response will increase the period of the foundation and most likely take it out of the low period reduced response range.

Note that response spectra curves in previous editions of AREMA showed a conservative flat region in the low period range. The accompanying commentary allowed for a low period reduced response spectral shape adjustment only if potential unaccounted sources of structure flexibility are considered. Given advancements in seismic hazard calculations as well as overall railroad bridge analysis practice it was felt that conservative flat region for low periods was no longer necessary. Rather, this provision provides caution for bridges in low period range and requires consideration of all potential sources of structure flexibility. This approach allows for better alignment with seismic hazard products being provided from sources such as USGS and GSC.

1.4.5.4 Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure may be used for two-span bridges or multi-span regular bridges as described in Article 1.4.5.2. The procedure is described below.

a. Calculate the Seismic Response Coefficient (C_m) for each of the two principal directions of the structure as follows.

   (1) Calculate the natural period of vibration (T_m) for each of the two principal directions of the structure using any commonly accepted method.

   (2) Calculate the Seismic Response Coefficient (C_m) for each of the two principal directions of the structure from Seismic Response Coefficient "Seismic Response Coefficient."

b. Perform static analysis on the bridge in each of the two principal directions.

   (1) Calculate the distributed seismic load in each direction from the following formula.

   $p(x)$ = distributed seismic load per unit length of bridge

   C_m = Seismic Response Coefficient

   $w(x)$ = distributed weight of bridge per unit length

   $$p(x) = C_m w(x)$$

   (2) Distribute the seismic load to individual members based on the stiffness and support conditions.

c. Combine the loads in each of the two principal directions of the structure to get the final seismic design loads.

   (1) Combination 1: Combine the forces in principal direction 1 with 30% of the forces from principal direction 2.

   (2) Combination 2: Combine the forces in principal direction 2 with 30% of the forces from principal direction 1.

C - 1.4.5.3 Equivalent Lateral Force Procedure

The Equivalent Lateral Force Procedure is included as a simple method of analysis that may be used for regular bridges. The calculations for this procedure are appropriate for hand calculation methods in most cases, though static computer analysis may be used to determine the load distribution to the individual members.

The two principal directions of the structure are typically the longitudinal and transverse directions of the bridge. For curved bridges, the longitudinal direction may be taken as a straight line connecting the centerline of the bridge at the beginning and end.

The natural period of vibration (T_m) for each of the two principal directions of the structure may be calculated using any commonly accepted method. The following simple formulation may be used.

$$T_m = 2\pi\sqrt{\frac{W}{gK}}$$

W= Total weight of the bridge.

g= Acceleration due to gravity (length/time^2)

K= The total structure stiffness including the stiffness of the superstructure, supporting members and surrounding soil.

The actual seismic response coefficient, C_m, varies throughout the structure in proportion to the relative lateral movement. A common method of equivalent lateral force analysis assumes that one-half the weight of the substructure is lumped at the superstructure level for the period calculation and the foundation load is calculated using the complete bridge weight with the seismic response coefficient determined for the superstructure. This analysis approach is accurate when the substructure weight is small relative to the superstructure weight, but may be too conservative for heavy pier substructures. Rather than using the more rigorous modal analysis approach, a simple modification to the equivalent lateral force procedure may be used to determine a less conservative foundation demand for bridges supported by heavy pier substructures. For single level bridges, it is conservative to assume that the actual seismic response coefficient, C_m, varies linearly from the peak ground acceleration (PGA) response coefficient at the ground level to the seismic response coefficient calculated at the superstructure level. Therefore, application of C_m on single level bridge substructures may be simplified by taking the average of the C_m value calculated in Paragraph 1.4.5.3a for the superstructure and the PGA response coefficient determined in accordance with Article 1.3.2.3 for the ground. However, this average C_m response shall never be taken as less than the PGA response coefficient.

The seismic load should be distributed to the individual members based on the stiffness and support conditions. For a regular structure with uniform weight per unit length and simple supports, this reduces to a simple beam calculation for the superstructure between supports and a single lateral load calculation for the supporting bents.

1.4.5.4 Modal Analysis Procedure

The Modal Analysis Procedure may be used for any structure configuration except complex bridge configurations as described in Article 1.4.5.2. The procedure is described below.

a. Develop the response spectrum from Seismic Response Coefficient "Seismic Response Coefficient."

b. Perform dynamic analysis on the structure in each of the two principal directions using the response spectrum to determine the individual member loads.

    (1) A mathematical model should be used to calculate the mode shapes, frequencies and member forces. The model should accurately represent the structure mass, stiffness and support conditions.

    (2) The structural responses should be calculated from an appropriate modal combination technique

    (3) An adequate number of modes should be included so that the response in each principal direction includes a minimum 90% mass participation.

c. Combine the loads in each of the two principal directions of the structure using one of the following methods to get the final seismic design loads.

    (1) SRSS Method - Combine forces in individual members using the square root of the sum of the squares from each principal direction.

    (2) Alternate Method - Perform two load combinations for investigation.

        (a) Combination 1: Combine the forces in principal direction 1 with 30% of the forces from principal direction 2.

        (b) Combination 2: Combine the forces in principal direction 2 with 30% of the forces from principal direction 1.

C - 1.4.5.4 Modal Analysis Procedure

The Modal Analysis Procedure is included as a general method of analysis that may be used for any bridge configuration except complex configurations. The calculations for this procedure are appropriate to be performed by any commonly available finite element computer program.

Response spectra used in the modal analysis procedure should have a well-defined spectral shape over the structure response period range of interest.

References

— 2024 —

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