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SEISMIC CODE ISSUES IN CENTRAL UNITED STATES

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SUMMARY

The Applied Technology Council (ATC), in a joint venture with Multidisciplinary Center for Earthquake Engineering Research (MCEER), has recently completed a project to develop recommended specifications and commentary for the seismic design of highway bridges (National Cooperative Highway Research Program (NCHRP) Project 12-49). These recommended specifications are being considered by the American Association of State Highway and Transportation Officials (AASHTO) for possible incorporation into the AASHTO LRFD Bridge Specifications (Capron et al., 2001). The purpose of this paper is to discuss the seismic hazard maps which are proposed to be used in this document as it impacts Tennessee. Various studies are compared with the USGS hazard maps and pros and cons of various methodologies are discussed.

INTRODUCTION

According to the NCHRP Project 12-49 or the Recommended *LRFD Guidelines for the Seismic Design of Highway Bridges: Part I Specifications* (hereafter will be referred to as NCHRP Specs), bridges shall be designed for the life safety level of performance. Life safety in the Maximum Credible Earthquake (MCE) event means that the bridge should not collapse, but complete or partial replacement may be required. The adopted hazard maps for the Rare Earthquake or MCE are the 3% probability exceedance in 75 years developed by the United States Geological Survey (USGS) research team (Frankel et al., 1996). The MCE corresponds to occurrence of an earthquake with a probability of exceeding 2% in 50 years. A probability of exceeding 2% in 50 years corresponds to a return period of approximately 2500 years.

COMPARISON OF RESULTS

In the last ten years, there has been significant progress made on several fronts in the development of seismic hazard maps. There has also been significant progress made in understanding seismic risks and associated uncertainties. The purpose of this paper is to present an overview of the background information required to understand seismic risks as they apply to Tennessee. This

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paper provides a comparative study among the USGS hazard maps for a 3% probability exceedance in 75 years, Pezeshk et al. (1998) acceleration coefficients for West Tennessee, and Toro and Silva (2001) hazard maps for the Central United States (CUS).

Frankel et al. (1996) provides a detailed description of the national probabilistic hazard maps. It should be noted that the USGS national hazard maps are for a 2% probability of exceedance in 50 years, which is in effect the same as a 3% probability of exceedance in 75 years. Seismic hazard maps reported by Toro and Silva (2001) for 3% probability of exceedance in 75 years at bedrock Toro and Silva (2001) utilized the current state of knowledge about earthquakes in the New Madrid (NM) and Wabash Valley seismic zones to develop seismic hazard maps for the Mississippi embayment. In the summary of their research, which was funded by USGS, Toro and Silva (2001) write, "The maps include the effect of depth and provide a level of detail superior to that of national maps." To provide a comparison among studies by Frankel et al. (1996), Pezeshk et al. (1998), and Toro and Silva (2001), acceleration values at 12 locations in West Tennessee were calculated and summarized in Table 1 for PGA, 0.2-second spectral acceleration, and 1.0-second spectral acceleration. The selected 12 locations were selected at intersections of 90, 89, 88 degree longitude and 35, 35.5, 36, and 36.5 latitude, respectively, to provide a broad representation of acceleration coefficients across West Tennessee. Figure 1 shows the acceleration coefficients at these 12 sites located on the map of West Tennessee.

From Table 1 and Figure 1, it can be observed that the newly calculated values for PGA, 0.2-second, and 1.0-second spectral accelerations by Toro and Silva (2001) are much smaller than the USGS acceleration values. Acceleration coefficients calculated by Pezeshk et al. (1998) in most cases fall between the USGS and the Toro and Silva (2000) values. Considering that Toro and Silva (2001) is the most recent study available where the current state-of-knowledge is used in developing hazard maps, it may be concluded that the U.S. National hazard maps may be conservative.

To understand why Frankel et al. (1996), Pezeshk et al. (1998), and Toro and Silva (2001) acceleration coefficients are different, it is important to understand how the seismic hazard at a site is determined. Seismic waves are initially generated at the fault in the earth. The waves propagate through the crust to the bedrock. Finally, the waves reach the ground surface through the soil layers above the bedrock. Three factors must be considered in evaluation of seismic hazard at a site: the seismic source(s), the propagation path of the seismic wave, and the local geology/site conditions.

From the deaggregation of seismic hazards in the Memphis area, it can be observed that the MCE ground motion is dominated by the characteristic earthquake used by Frankel et al. (1996) in producing the 1996 hazard maps. Frankel et al. (1996) used a characteristic earthquake with a moment magnitude **M** 8.0 with a recurrence interval of 1000 years for the NM fault. However, there are many uncertainties involved in the determining the magnitude and the recurrence interval considered in developing the national hazard maps.

The following sections provide a detailed discussion concerning specific aspects of developing seismic hazard maps.

Table 1. Comparison of Acceleration Coefficients.

Coordinates		National Hazard Maps			Pezeshk et al. (1998)			Toro and Silva (2001)		
Latitude	Longitude	PGA	0.2 sec	1 sec	PGA	0.2 sec	1 sec	PGA	0.2 sec	1 sec
35.0	90.0	52.0	112.0	32.0	22.3	58.0	22.5	18.0	24.5	12.0
35.5	90.0	116.0	244.0	75.0	79.1	115.0	51.0	45.0	63.0	26.0
36.0	90.0	151.0	333.0	99.0	77.6	143.0	46.0	75.0	140.0	48.0
36.5	90.0	105.0	212.0	64.0	70.7	112.0	42.8	50.0	70.0	30.0
35.0	89.0	26.0	59.0	21.0	14.0	38.0	16.3	12.0	16.1	8.0
35.5	89.0	43.0	97.0	28.0	15.6	38.0	18.0	19.0	25.9	13.0
36.0	89.0	80.0	155.0	45.0	25.5	56.0	23.0	20.0	63.0	28.0
36.5	89.0	88.0	168.0	48.0	48.5	79.0	33.0	17.5	56.0	28.0
35.0	88.0	15.0	38.0	17.0	9.5	18.3	14.7	7.0	9.8	6.5
35.5	88.0	20.0	49.0	19.0	6.4	17.5	16.3	9.0	16.2	7.0
36.0	88.0	28.0	61.0	21.0	5.5	35.6	19.4	13.0	17.5	8.5
36.5	88.0	38.0	86.0	25.0	8.8	27.5	20.0	19.0	27.3	11.0

All acceleration values are in %g.

Magnitude

The 1811-1812 earthquakes in the NM seismic zone (NMSZ) represent a historically unique sequence of large intraplate seismic events that included more that 18 earthquakes with magnitudes of 6.0 or higher (Johnston and Schweig, 1996). However, there is considerable uncertainty about the magnitude, and the location and dimensions of the source zones of the principal events (Crone, 1998 and Gomberg, 1996). Frankel et al. (1996) used a characteristic rupture model with a characteristic moment magnitude **M** 8.0 as the estimated magnitude of the largest events in the 1811-1812 sequence based on the study by Johnston (1996). However, in several other research papers (Hough, et al., 1999 and Wheeler and Perkin, 2000), it has been noted that the 1811-1812 earthquakes had moment magnitudes in the range of 7.1 to 7.5. Both Toro and Silva (2001) and Pezeshk, et al. (1998) considered moment magnitudes smaller than 8.0 in their studies. Similarly, Speidel (1998) and Newman et al. (1999) suggest that the moment magnitude of the characteristic earthquake is approximately 7 instead of 8 (Hwang, 2000). However, according to Wheeler and Perkins (2000), large uncertainties associated with the methodology used by Newman et al. (1999) may make their conclusions questionable. The uncertainties stem from the fact that the 1811-1812 earthquakes occurred in a sparsely populated area prior to the development of seismographs. Therefore, using a moment magnitude **M** 8.0 in developing the national hazard maps for the NM area seems conservative.

Recurrence Interval

There are uncertainties involved in selecting a recurrence interval for the characteristic earthquake in the NM area. Frankel et al. (1996) used a recurrence of 1000 years for the magnitude 8.0 characteristic earthquake based on paleoliquefaction evidence of past earthquakes. However, Johnston and Schweig (1996) and Schweig and Tuttle (1999) suggest a 500-year recurrence time. Conversely, Newman et al. (1999) suggest a much longer recurrence time than 1000 years.

Source Location

Seismic hazard analysis is sensitive to the location of seismic source zones or faults and their seismicity parameters. In the CUS, there are many uncertainties in delineating seismic source zones and determining seismicity parameters due to low seismic activity and the lack of complete knowledge about the cause of earthquakes. The geometry and the location of the NM fault are not well known. Due to the lack of fault traces on the ground, it is difficult to determine the fault rupture length and location. To consider uncertainties of seismic source zones and seismicities of each zone, Pezeshk et al. (1998) estimated the seismic hazard for West Tennessee on the basis of different experts' opinion about the seismic source zone and seismicity parameters. Consequently, Pezeshk et al. (1998) used three seismic source zone sets and corresponding parameters based on experts' opinion. Frankel et al. (1996) used the spatially-smoothed seismicity as one component of the hazard calculation. This assumes that future damaging earthquakes will occur near areas which have had smaller earthquakes of moment magnitude $M>3$ or $M>4$ (Frankel et al., 1997). In addition, Frankel et al. (1996) used large background zones based on broad geologic criteria to quantify hazards in areas with little or no historic seismicity but with the potential for damaging earthquakes. Toro and Silva (2001) have done an extensive amount of work to implement the most recent fault locations in developing their seismic hazard maps.

Attenuation

Another source of uncertainty is attenuation models. For the purpose of engineering applications such as seismic hazard analysis, ground motion is estimated according to an attenuation equation. Because an attenuation function of ground motion is determined from the statistical analysis of strong ground motion, the data should contain a good distribution of both magnitude and distance. However, in the NM area, there are only a few small to intermediate earthquakes with limited data recordings. Therefore, it is impossible to estimate the ground motion attenuation function from recorded data. To overcome the difficulty, ground motion attenuation functions for the NM area are based on artificially generated ground motion.

Earthquake ground motion is related to the seismic magnitude, epicenteral distance, and local site conditions. The attenuation equation should include magnitude, distance, and site conditions. At the same time, there are uncertainties of ground motion for given magnitudes and distances, as discussed above, due to the complexity of seismic source and wave propagation paths. To best-fit observations, past researchers have proposed various formulations.

Atkinson (1984) was the first to estimate ground motion attenuation for Eastern North America (ENA) using the stochastic-point-source model and random vibration theory. EPRI (1986) and Boore and Atkinson (1987) also proposed different attenuations for the ENA. Other relationships are those of Atkinson and Boore (1995) and Toro et al. (1997). Atkinson and Boore (1995) used an empirical-source model while Toro et al. (1997) used Brune's point-source model. Toro et al. (1997) also considered parametric uncertainties in stress parameters, in the near site attenuation factor *k*, and in the quality factor *Q(f)*.

Past experience shows that ground motion attenuations are different for intermediate and large earthquakes. Geometric and anelastic wave attenuations for distances less than 100 km are different from those for distances greater than 100 km in the ENA. To consider all these factors, following EPRI (1993), Pezeshk et al. (1998) developed an attenuation relation. Pezeshk et al. (1998) used two independent quality factors, one from Atkinson and Boore (1995) and another from Dwyer (1984). Toro and Silva (2001) developed a new attenuation relationship for the CUS. In developing their attenuation relationship, Toro and Silva (2001) assumed that stress-drop decreases with increasing magnitude for the single-corner model. Because of this, the CUS single-corner model shows lower peak accelerations, particularly at large magnitude, than the Toro et al. (1997), EPRI (1993), and Pezeshk et al. (1998) relations.

Frankel et al. (1996) used two equally weighted, attenuation relations. First, they used the attenuation equation developed by Toro et al. (1993) based on m_{blg}. The attenuation relations were multiplied by frequency-dependent factors to convert them from hard-rock to firm-rock sites. Second, they used a set of relations derived by Frankel and others for firm-rock sites. Frankel et al. (1996) used a constant stress drop of 150 bars. Toro et al. (1997) used a stress drop of 120 bars, and Toro and Silva (2001) used a stress drop of 160, 120, and 95 bars for magnitudes of moment magnitudes **M** 5.5, 6.5, 7.5, respectively.

There is a great of amount of uncertainty in which attenuation relationships should be used in developing seismic hazard maps. Wheeler and Perkins (2000) state, " … the need to add the Atkinson-Boore 1995 model for the 2001 (USGS) maps, with a weight of at least 1/3." This points to the fact that the 1996 national seismic hazard maps may have deficiencies which are being addressed in the 2002 hazard maps.

Return Period

In a publication by USGS (2002) under the topic of "How likely is a New Madrid earthquake?" it mentioned that the probability of a repeat of 1811-12 earthquakes (magnitude 7.5-8.0) is between 7 and 10 percent. Similarly, the probability of a magnitude 6.0 or larger is between 25 to 40 percent. Using a simple formulation, one can obtain the return period of an earthquake with magnitude 7.5-8.0 to be between 475 to 690 years and for a magnitude 6 to be between 98 to 174 years. Therefore, it seems that using a return period of 2,500 years (3% in 50 years) is excessive.

Based on the discussion in NCHRP 12-49 Appendix A, the reason a return period of 2,500 years was selected is "If deterministic estimates of ground motions are made for historic New Madrid earthquake of estimated moment magnitude 8.0 using the same ground motion attenuation relationships used the USGS probabilistic ground motions mapping, then the 500-year mapped ground motion are at below the deterministic median-minus standard-deviation ground motions estimated for historic events …" In other words, a return period of 2,500 year was selected to be above the deterministic median-minus standard deviation ground motions. However, this is not true any more. Due to the recent finding of the USGS (2002) the return period of magnitude 8.0 earthquake in the New Madrid is about 435 years. Therefore, there is no need to design for 2,500 years to satisfy this requirement.

It is obvious from the previous discussions that there are many unresolved issues in developing seismic hazard maps in the NM area. In addition to the uncertainties involved, another important issue is whether an earthquake with a return period of 2500 years is appropriate for West Tennessee. Hwang (2000) suggests, "the maximum considered earthquake specified in the NEHRP Provisions does not need to be an earthquake with a return period more than 1000 years. In other words, it is not necessary to define an earthquake with a return period of 2500 years as the basis for the design of ordinary buildings." Whether to use a 1000 or a 2500-year return period is an important issue that must be considered carefully.

Hazard Curves

U.S. National Seismic Hazard Maps are constructed from mean hazard curves, which consider the mean probabilities of exceedance as a function of ground motion or spectral response (Frankel, 1996). Seismic hazard curves for various cities in the United States are different. Figure 2 shows the hazard curves for San Francisco, Los Angeles, and Memphis under a reference firm-rock site. From Figure 2, one can observe the general difference in the slope of hazard curves for the CUS and Western United States (WUS). As a reference, 10% probability of exceedance in 50 years (500 year return period) corresponds to an annual frequency of exceedance of $2.1x10^{-3}$ and 2% in 50 years corresponds to $4.04x10^{-4}$. Using return periods of 500 and 2500 years, the PGA for Memphis, San Francisco, and Los Angeles have been determined and summarized in Table 2. In addition, in Table 2, for presentation purposes only, the acceleration values for the MCE (2500-year return period) are multiplied by 2/3 to determine design acceleration levels as suggested by NEHRP for design of buildings (BSSC, 1998). In Table 2, the difference between the 500-year return period design and 2500-year return period for San Francisco and Los Angeles is negligible. However, for the Memphis area, the increase in design acceleration values from a 500-year return period to 2500-year return period is 3.13. Therefore, the 500-year and 2500-year hazard maps for both cities in California will result in virtually the same level of seismic forces. In Memphis, the level of seismic forces will be increased by factor of 3.13. Hwang (2000) found that for a 500-year return period, the expected ground motion in Memphis is only about one-quarter (24%) of that in San Francisco. However, for a 2500-year return period, the expected ground motion in Memphis is increased to approximately the same level (81%) of that in San Francisco.

Table 2. Comparison of Seismic Hazard (PGA) in Memphis, San Francisco, and Los Angeles.

Return Period (Years)	**2500 (1)**	**500 (2)**	**Design (3)**	**Ratio (3)/(2)**
Memphis	0.72	0.15	0.48	3.13
San Francisco	0.88	0.58	0.58	1.01
Los Angels	0.75	0.49	0.50	1.02

Site Response

According to the NCHRP Specs Section 3.4.1, design response spectra for the MCE and the expected earthquake shall be constructed using the accelerations from national ground motion maps. Design earthquake response spectral accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are determined by:

$$S_{DS} = F_a S_s$$

and

$$S_{D1} = F_V S_1$$

where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps and F_a and F_v are site coefficients as given in Tables 3 and 4, respectively. Site coefficients F_a and F_v depend on the Site Class as defined in Table 5.

According to NCHRP Specs Table 3.4.2-1 (Table 5 given below) titled "Site Classification," a site class B is a rock site with average shear wave velocity of top 100 ft (30 m) of soil to be 2500 ft/s < V_s< 5000 ft/s or 760 m/s < V_s< 1500 m/s. Similarly, a site class C is a very dense soil and soft rock with 1200 ft/s < V_s< 2500 ft/s or 360 m/s < V_s< 760 m/s. Therefore, a B-C boundary would be a zone in a soil profile with shear wave velocity transitioning from above 2500 ft/sec (760 m/sec) to below 2500 ft/sec (760 m/s). In West Tennessee, in particular, in Memphis, the B-C boundary is located approximately 1000 ft below the ground surface (Pezeshk et al., 1998). Therefore, using site coefficients F_a and F_v, which are for only the top 100 ft of soil, is not appropriate in West Tennessee because the B-C boundary in Memphis is not at 100 ft, it is at approximately 1000 ft below the ground level. Following the NCHRP Specs is not accurate because 900 ft of soil, which lies between the top 100 ft and the B-C boundary, is ignored. Site-specific studies have been performed by both Toro and Silva (2001) for Memphis and St. Louis, and by Pezeshk et al. (1998) for the West Tennessee area to overcome the problem.

Since Pezeshk et al.'s (1998) study, several advances have been made in the assessment of seismic hazard including the development of a new one-dimensional non-linear wave propagation model to evaluate the amplification/attenuation characteristics of the deep soil deposits. The newly developed model is specifically designed to account for the effect of confining pressure on shear modulus and damping characteristics of soil deposits. The new model shows that the propagated motions are significantly higher than would be obtained using conventional models (Hashash and Park, 2001; Park, et al. 2004). Preliminary analyses using this model show that there is significant amplification of long period waves through the thick deposits of the embayment. The deep deposits are capable of transmitting some high frequency components of the ground motion as well. Therefore, previous studies need to be revisited to update using the new research advances.

FINAL COMMENTS

This paper provides a few comments regarding the seismic hazard maps that will be used in the AASHTO LFRD Bridge Specifications. Differences among three studies (Frankel et al., 1996, Pezeshk et al., 1998, and Toro and Silva, 2001) have been discussed. In the CUS, there are multiple uncertainties involved in determination of magnitude, attenuation relations, fault and seismic source zones, and recurrence intervals. There is also the issue of site characterization, which plays an important role in the CUS.

There are great differences in national seismic hazard maps, and the ones developed by Pezeshk et al. (1998) and Toro and Silva (2001). Toro and Silva (2001) produced the most recent study, in which the most recent state of knowledge is used, indicates that the national seismic hazard maps may be conservative for the West Tennessee area.

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

Site	Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	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	a
F	a	a	a	a	a

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

Site	Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1 Second Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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	A
F	a	a	a	a	a

Table 5. Site Classification.

Soil Profile Type	**Description**	$\overline{V}_s$ **(ft/sec)**	$\overline{S}_u$ **(psf)**
A	Hard Rock	$\overline{V}_s > 5000$	
B	Rock		
C	Very Dense Soft Rock $\overline{N} > 50$	$2500 < \overline{V}_s \leq 5000$	$\overline{S}_u \geq 2000$
D	Stiff Soil $15 < \overline{N} < 50$	$1200 < \overline{V}_s \leq 2500$	$1000 < \overline{S}_u \leq 2000$
E	PI>20 W> 40	$600 < \overline{V}_s \leq 1200$	$\overline{S}_u \leq 500$
F	Need Site Specific Study		

S_u=undrained Shear Strength
PI = Plastic Index
N = Standard penetrationResistance

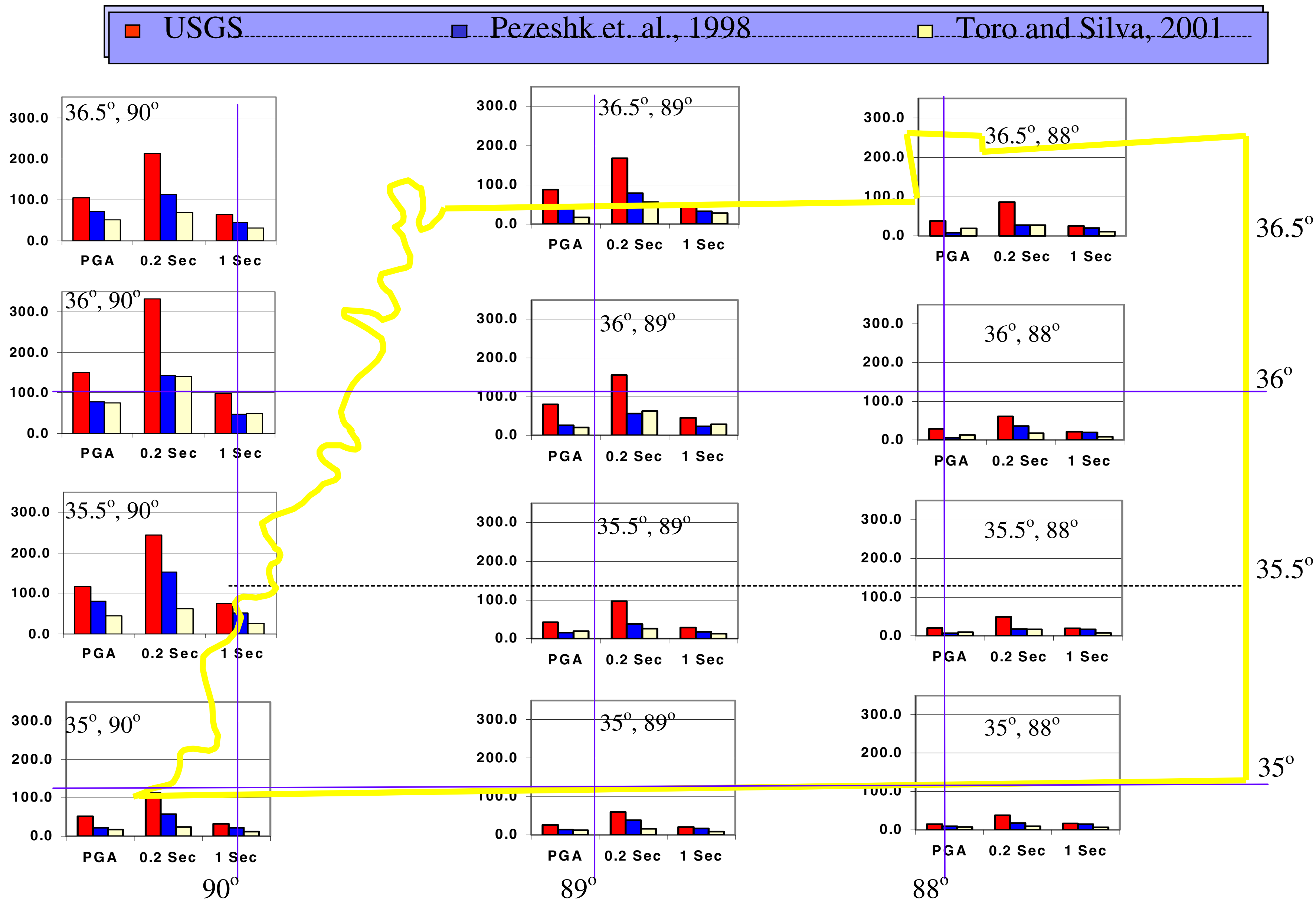
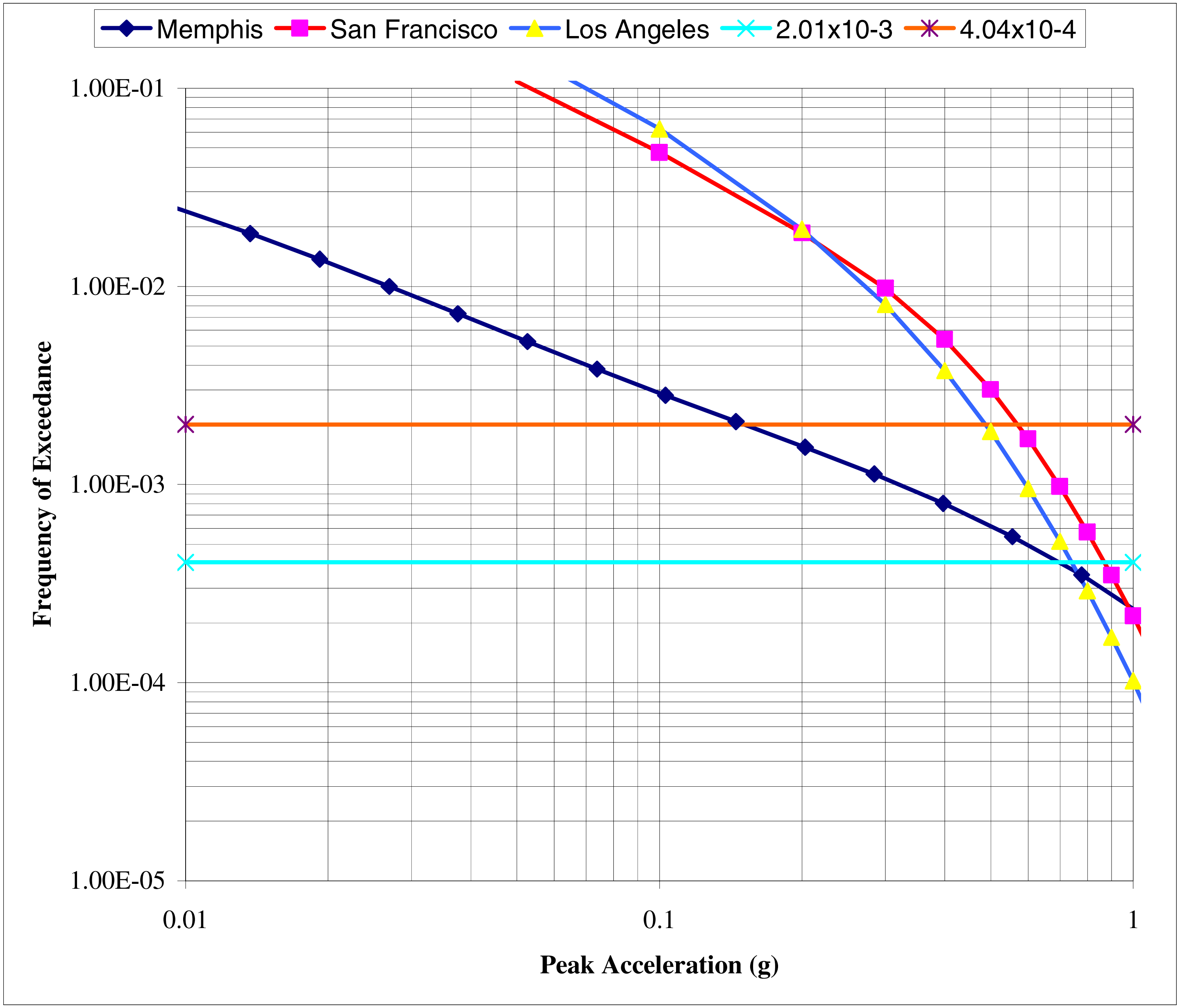


Figure 1. Comparison of PGA, 0.2-Second Spectral, and 1.0-Second Spectral Acceleration Coefficients from Three Different Studies.



Memphis
San Francisco
Los Angeles
2.01x10-3
4.04x10-4
1.00E-01
1.00E-02
1.00E-03
1.00E-04
1.00E-05
0.01
0.1
1
Frequency of Exceedance
Peak Acceleration (g)

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