

## APPENDIX H

### SEISMIC RISK ANALYSIS OF A MULTISPAN BRIDGE

NCHRP Project 12-49 has recently proposed a set of recommended load resistance factor design (LRFD) guidelines for the seismic design of highway bridges, published as *NCHRP Report 472: Comprehensive Specifications for the Seismic Design of Bridges* (Applied Technology Council [ATC] and the Multidisciplinary Center for Earthquake Engineering Research [MCEER], 2002). The proposed LRFD guidelines have adopted and modified many of the features outlined by the existing National Earthquake Hazards Reduction Program specifications (NEHRP, 1997), which were originally intended for buildings, to the design and safety evaluation of bridge structures. Thus, for the life safety limit state, the proposed LRFD seismic guidelines use the NEHRP 2,500-year return period earthquake hazard maps along with the response spectra proposed by NEHRP, but remove a  $\frac{2}{3}$  reduction factor that is associated with the NEHRP spectral accelerations. The  $\frac{2}{3}$  factor had been included by NEHRP to essentially reduce the 2,500-year acceleration spectrum to an equivalent 500-year spectrum for the U.S. West Coast region.

Another major change proposed by NCHRP Project 12-49 consists of a new set of response modification factors that more realistically model the nonlinear behavior of bridge components. Specifically, NCHRP Project 12-49 has proposed higher response modification factors for multicolumn bents, which would reduce the implicit levels of safety in the design process and may offset the use of the more conservative acceleration spectrum. In addition, NCHRP Project 12-49 proposed a performance-based design approach whereby different seismic design and analysis procedures (SDAPs) are specified, depending on the seismic hazard level and whether the bridge is expected to perform adequately for a life safety or operational performance objective (ATC and MCEER, 2002).

The analyses performed in the previous appendixes were executed for bridge piers having foundations formed by pile extensions by modeling the bridge pier as an equivalent single degree of freedom (SDOF) system after determining the effective point of fixity. In this appendix, the analysis of an example bridge founded on a pile system, which had been previously prepared by the NCHRP Project 12-49 team, is reviewed and is used as the basis for the seismic risk analysis. This particular analysis is based on a more advanced multi-degree of freedom (MDOF) structural analysis. The objectives of this appendix are (1) to compare the results of the risk analysis obtained using simple SDOF systems with those that using an MDOF spectral analysis and (2) to compare the safety levels of bridge bents founded on multiple piles with those of bridge bents founded on pile extensions.

#### 1. DESCRIPTION OF EXAMPLE BRIDGE

To achieve the objectives of this appendix, an example bridge that was used as part of NCHRP Project 12-49 to illustrate the recommended analysis and design methods is selected for review. Specifically, the bridge selected for this analysis is the 500-ft bridge described in the unpublished Design Example No. 8 developed under NCHRP Project 12-49. The original site was taken by the NCHRP Project 12-49 team to be in the Puget Sound region of Washington state. However, in this appendix, the analysis is repeated for the seismic zone corresponding to Seattle. The change in site location is effected to take advantage of the availability of the Seattle seismic intensity data. The United States Geological Survey (USGS) earthquake hazard maps indicate that, for the Seattle site, the short-period acceleration (0.2 sec) is  $S_s = 1.61$  g and the 1.0-sec acceleration is  $S_1 = 0.560$  g for the most credible earthquake (MCE) corresponding to the earthquake with a 2,500-year return period.

The 500-ft bridge is formed by five continuous spans of 100 ft each. The four monolithic bents are formed by two columns, each integrally connected to the cap of the combined concrete piles with steel casings that form the foundation system. The two-column bents are also monolithically connected to the box girder superstructure through a crossbeam. Expansion bearings form the connections between the superstructure and the substructure at the two end stub-type abutments. Figures 1.a, 1.b, and 1.c—which are adopted from the NCHRP Project 12-49 example—provide a description of the bridge.

The NCHRP Project 12-49 team chose to use the commercial structural analysis program SAP2000 to perform a multimode spectral analysis of the example bridge. The structural model for each bent is represented by Figure 2, which is adapted from the unpublished NCHRP Project 12-49 report, in which the effect of the foundation is modeled by a series of translation and rotational springs. The same model used in the NCHRP project report is used in this appendix, which focuses on examining the safety of the bridge columns when the bridge is subjected to earthquakes having intensities similar to those expected in Seattle. A separate analysis is executed for each of the transverse and longitudinal directions because the uncertainties associated with the direction of the earthquake and the use of different possible combination rules are beyond the scope of this study.

#### 2. ANALYSIS OF EXAMPLE BRIDGE

The bridge example described in the previous section was analyzed using the same input data for the structural model

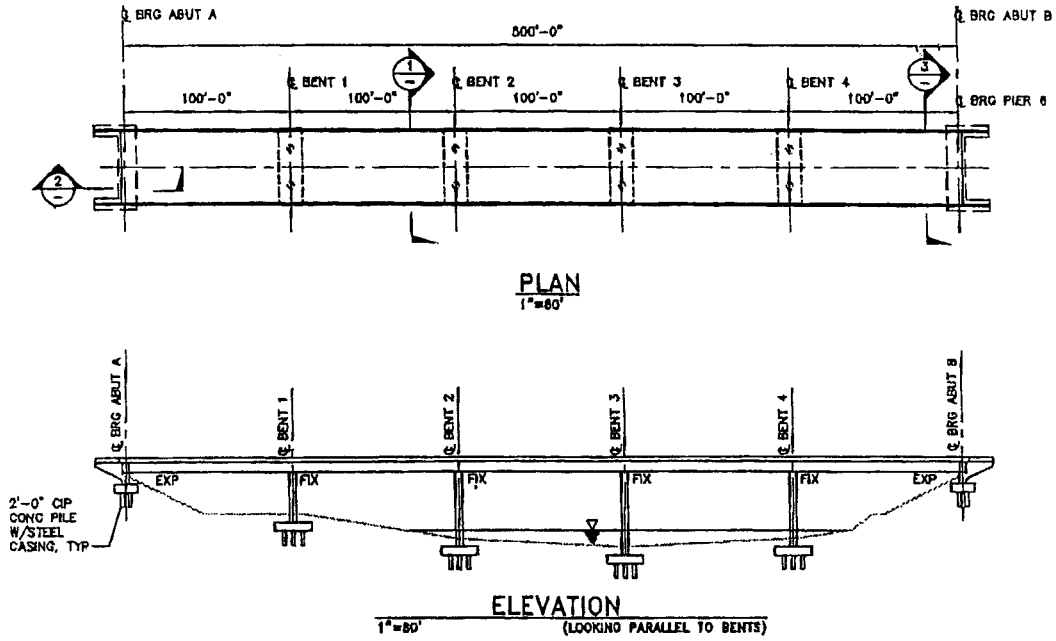


Figure 1.a. Plan view of example bridge.

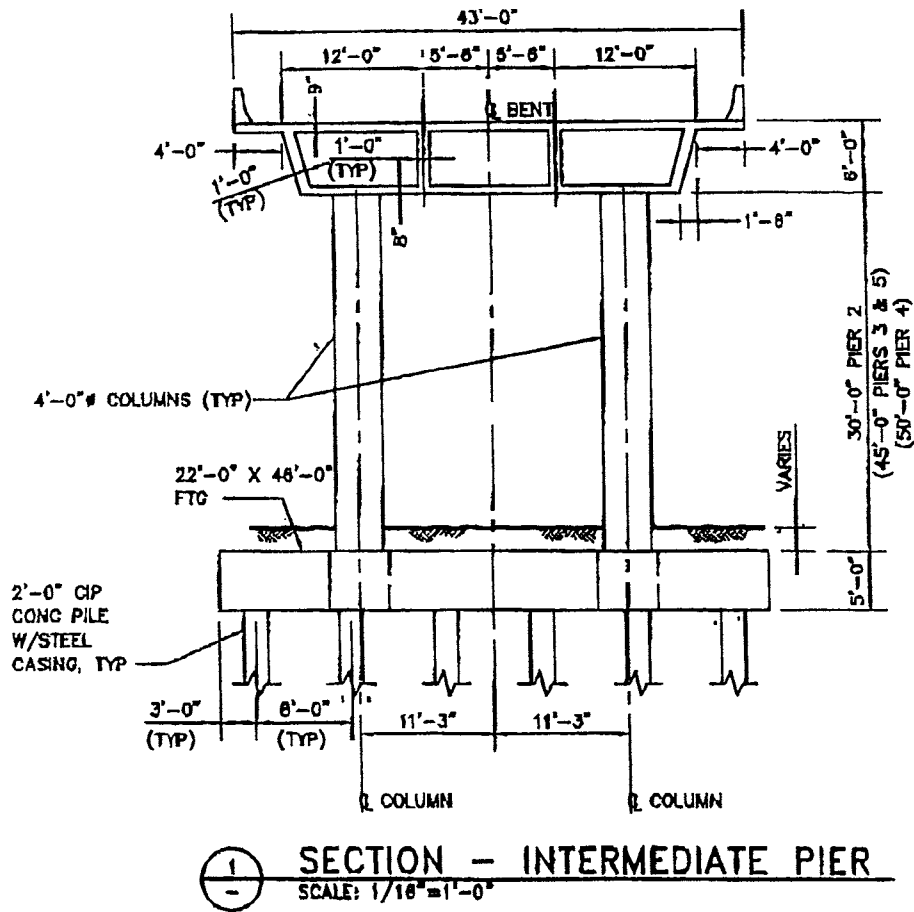


Figure 1.b. Cross section of example bridge.

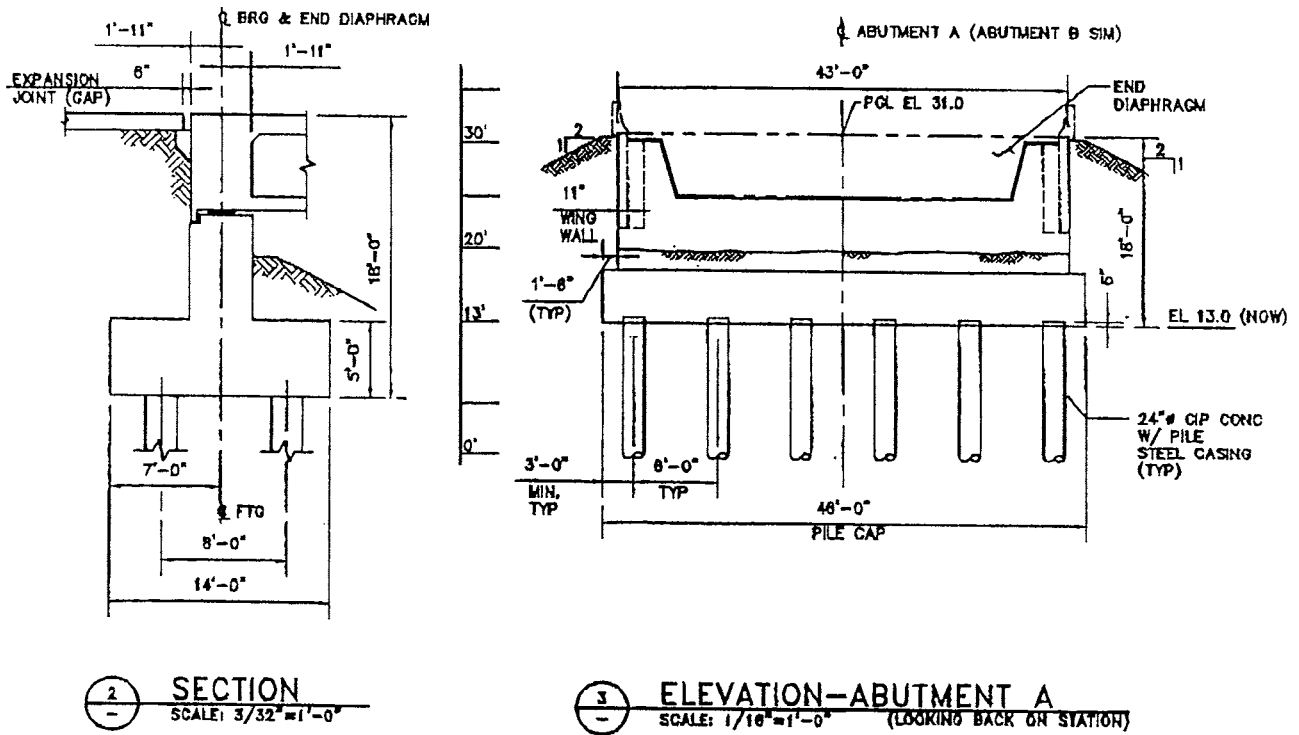


Figure 1.c. Profile of bridge example abutment.

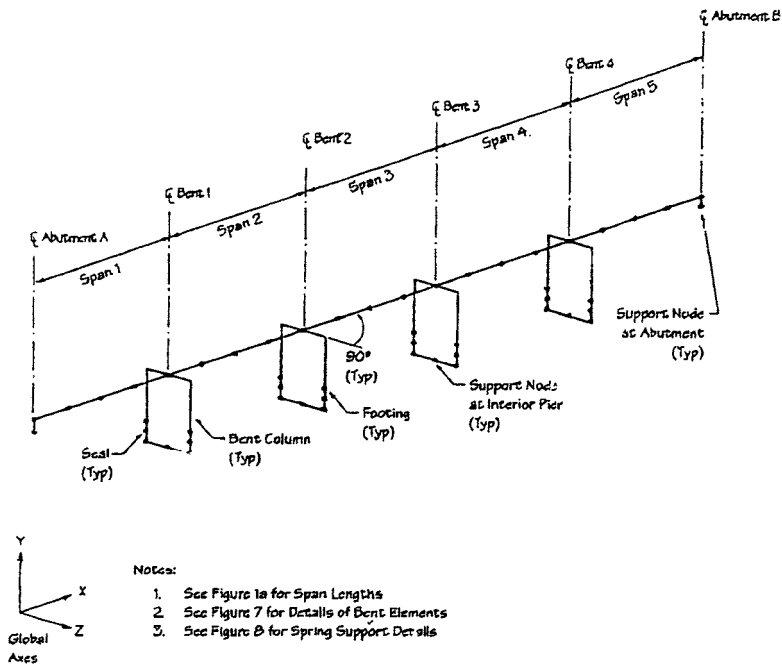
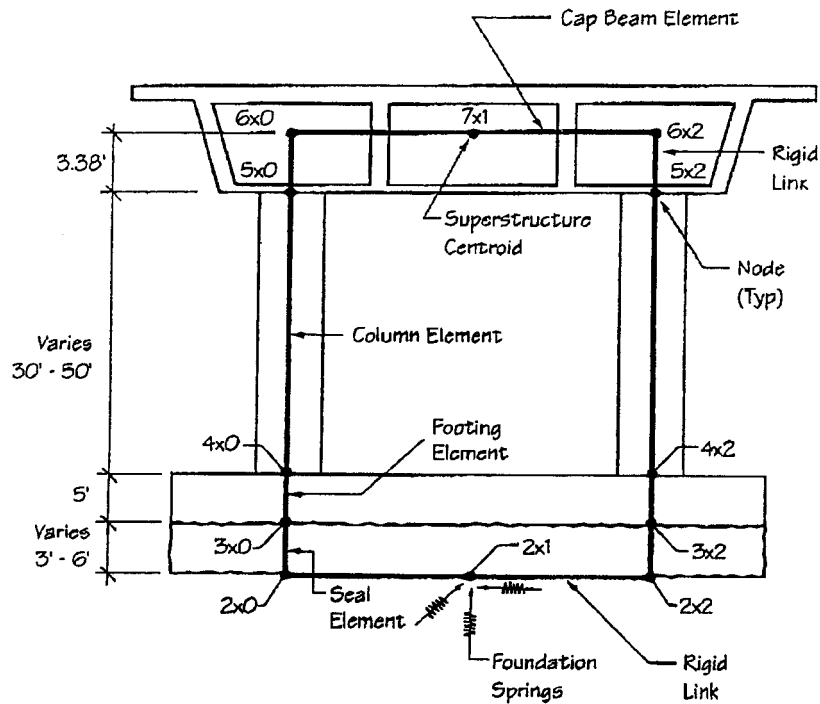


Figure 2.a. Model for structural analysis of example bridge.



Notes:

- 1.) Looking Ahead on Line
- 2.) "x" Represents Pier Number (Pier 2, 3, or 5)

Figure 2.b. Structural model for column bent with foundation springs.

that was previously used by the NCHRP Project 12-49 team, but using the earthquake response spectrum corresponding to the MCE of the Seattle site. The MCE, as explained by NEHRP (1997) and ATC and MCEER (2002), corresponds to the earthquake with a 2,500-year return period. On the other hand, the AASHTO LRFD specifications specify that bridges should be designed for a 75-year design life. Hence, the difference between the 2,500-year design earthquake return period and the 75-year design life provides an implicit level of safety that must be taken into consideration while performing a seismic risk analysis or when calculating the reliability index for the bridge under consideration. Similarly, the use of response modification factors that are lower than those observed during experimental investigations of bridge members provide additional safety factors. Such implicit safety factors compensate for the use of a load factor equal to  $\gamma_{EQ} = 1.0$ , which is stipulated by the AASHTO LRFD seismic specifications and other seismic design codes.

The SAP2000 structural model is presented in Figures 2.a and 2.b. The results of the SAP2000 analysis using the MCE for the Seattle site are summarized in Table 1, which shows the maximum moments produced in the bridge columns assuming that the *EQ* is applied independently in the longitudinal or the transverse directions of the bridge. Using a response modification factor of  $R_m = 6.0$ , as is done in the NCHRP Project 12-49 example, indicates that the bridge col-

umns should be designed to have moment capacities exactly equal to  $M_{x\text{design}} = 1,606$  kip-ft ( $= 9638.5$  kip-ft/6) for bending about the *x* axis and  $M_{z\text{design}} = 2,800$  kip-ft ( $= 16,800$  kip-ft/6) for bending about the *z* axis.

Following traditional practice, the design process outlined above and described in detail by ATC and MCEER (2002) treats all the variables that control the seismic safety of the bridge as deterministic variables. However, it is well known that a large number of uncertainties including modeling and inherent uncertainties are associated with the seismic analysis, design, and bridge construction processes, as well as the estimation of future loads and the identification of material properties. These uncertainties imply that bridges designed and constructed following current procedures are associated with a certain level of risk to failure within their intended 75-year design life. A model to estimate this level of risk as measured by the reliability index,  $\beta$ , is presented in the main body of this report (*NCHRP Report 489*). The model considered that the dominant uncertainties controlling the seismic risk of bridge systems may be included into five random variables that are related to the following:

1. Expected maximum earthquake intensity for the bridge site within the bridge's design life;
2. Natural period of the bridge system;

**TABLE 1 Summary of SAP2000 results for forces and moments in bridge columns**

	Axial Force (kips)	Moment X (kip-ft)	Shear Z (kip)	Moment Z (kip-ft)	Shear X (kip)
<i>EQ Longitudinal</i>					
Bent 1 column	39	0	0	16,800	1,119
Bent 2 column	56	0	0	7,661	340
Bent 3 column	11	0	0	6,239	249
Bent 4 column	33	0	0	7,688	342
<i>EQ Transverse</i>					
Bent 1 column	723	7,544	499	1,089	73
Bent 2 column	778	8,028	355	825	37
Bent 3 column	899	9,639	383	295	12
Bent 4 column	790	8,675	383	785	35

3. Spectral accelerations for the site taking into consideration the soil properties;
4. Nonlinear behavior of the bridge system; and
5. Modeling uncertainties associated with current methods of analysis.

Information available in the literature about these random variables are discussed in detail in the main body of this report and are summarized in the first six rows of Table 2 of this appendix. The distribution of the earthquake accelerations for different sites are also shown in Figure 3, which is based on data provided by USGS and Frankel et al. (1997).

As mentioned above, the structural and reliability analyses used in this report are based on an SDOF model of a bridge

bent. The use of a multimodal analysis for this bridge example implies that the analysis is based on several vibration modes. Therefore, the consideration of a single random variable, namely  $t'$ , to represent the uncertainties associated with determining the natural period of the bridge is no longer possible. An alternative approach consists of recognizing that the period of the system is a function of the stiffness of the structural members and those of the foundation. The stiffness of each member is controlled by the product of the modulus of elasticity and the moment of inertia. If one assumes that the moment of inertia is deterministic, then the stiffness of the structural member is controlled by one random variable, namely the modulus of elasticity,  $E$ . The nominal modulus of elasticity used in design practice is the value obtained for

**TABLE 2 Summary of input values for seismic reliability analysis**

Variable		Bias	Coefficient of Variation	Distribution Type	Reference
Earthquake modeling, $\lambda_{eq}$		1.0	20%	Normal	Ellingwood et al. (1980)
Spectral modeling, $C'$		1.0	Varies per site (15% to 40%)	Normal	Frankel et al. (1997)
Acceleration A	San Francisco	1.83% g (yearly mean)	333%	from Figure 3	USGS website
	Seattle	0.89% g (yearly mean)	415%		
	Memphis	0.17% g (yearly mean)	1707%		
	New York	0.066% g (yearly mean)	2121%		
	St. Paul	0.005% g (yearly mean)	3960%		
Period, $t'$		0.90	20%	Normal	Chopra and Goel (2000)
Weight, $W$		1.05	5%	Normal	Ellingwood et al. (1980)
Response modification, $R_m$		7.5 (mean value)	34%	Normal	Priestly and Park (1987); Liu et. al (1998)
Modulus of elasticity, $E$		1.25	40%	Normal	Deduced from Chopra and Goel (2000)
Foundation spring stiffness, $K_s$		1.0	17%	Normal	Stewart et al. (1999)

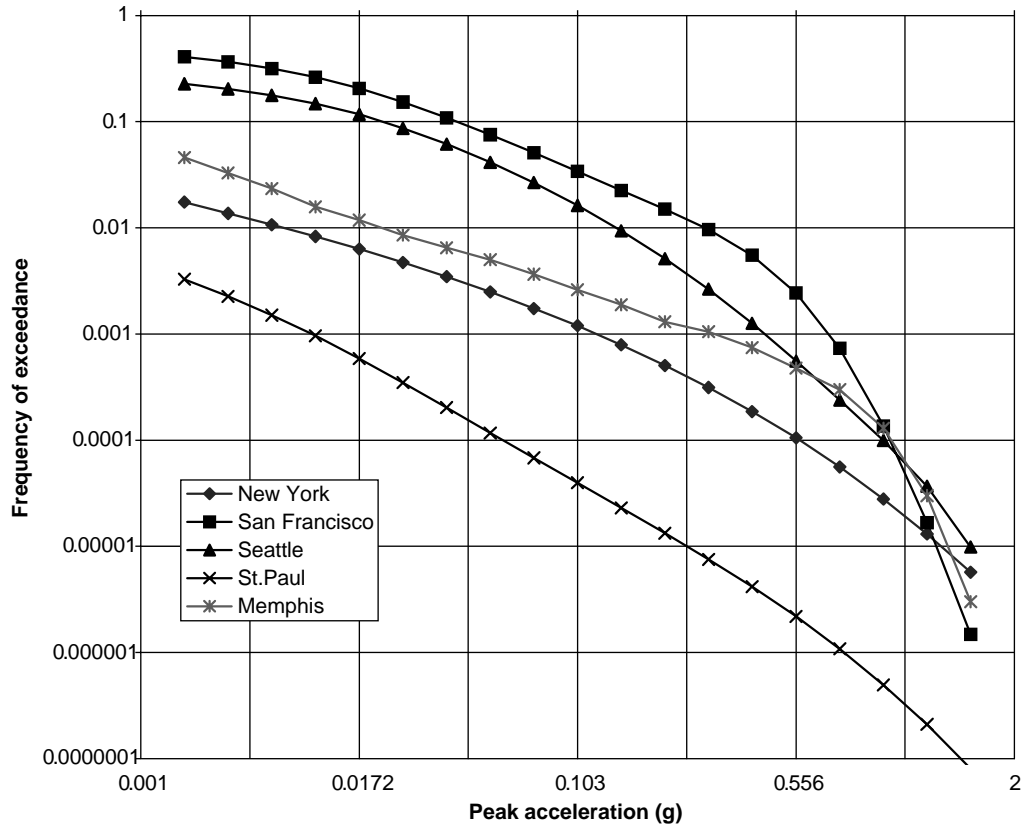


Figure 3. Annual probability of exceedance curves for peak ground acceleration [based on USGS website and Frankel et al. (1997)].

concrete at 28 days. However, some researchers have assumed that the actual long-term modulus for concrete members is on the order of 1.25 times that of the nominal value. For this reason, in the calculations performed in this example, the mean value of  $E$  is assumed to be 1.25 times the nominal value. This bias will also result in a bias approximately equal to 0.90 for the natural period of an SDOF system, which corresponds to the period bias used in this report. Using similar logic, it is proposed to use a coefficient of variation (COV) of  $V_E = 40\%$  for the modulus of elasticity during the reliability analysis of this bridge example. In fact,  $V_E = 40\%$  would result in a COV of 20% for the period of an SDOF system, which is the same value used during the analyses performed in other appendixes of this report. To account for the uncertainties associated with determining the spring stiffness values of the foundation system, the data collected by Stewart et al. (1999) are used. These data indicate that models used to estimate foundation spring coefficients are associated with a bias equal to 1.0 and a COV on the order of 17%.

This report used a closed-form expression to model the SDOF response of bridge piers to earthquake accelerations. Such a closed-form solution lent itself to the use of full-fledged Monte Carlo simulations to study the risk to failure and calculate the reliability index for a variety of bridge configurations. The fact that the response of this bridge example

is not based on an explicit formulation but is obtained implicitly through the use of a commercial analysis program—which requires the interference of the analyst to prepare the input data and interpret the results—precludes the use of the Monte Carlo simulation that requires an extremely large number of analyses and requires the use of more approximate techniques. In particular, the Response Surface Method (RSM) has been used by several researchers to incorporate structural analysis results into structural reliability computations. RSM is based on artificially constructing an equation to model the response of the structure using a polynomial fit to the results obtained from a limited number of discrete numerical analyses. Melchers (1999) observes that when the approximating surface fits the point responses reasonably well in the area around the most likely failure point, RSM provides a good estimate of the reliability index,  $\beta$ . Based on these observations and the difficulties associated with using a Monte Carlo simulation, RSM is used in this report to obtain estimates of the reliability index for the bridge example described above.

The first step in RSM consists of using the structural analysis program SAP2000 at various values of the random variables and of studying the response of the bridge for these input parameters. Melchers (1999) and others (e.g., Ghosn et al., 1994) have indicated that the best points to use during these analyses should be selected such that the points lie close to

the most likely failure point. However, since the failure point is not known a priori, an iterative process must be used. In this case, the analysis is first performed at points lying close to the mean values (mean  $\pm 1$  standard deviation) and additional refinements are introduced following the estimation of the location of the most likely failure point. Because the earthquake intensity is associated with an extremely high standard deviation, the SAP2000 analysis is performed at the mean 75-year earthquake acceleration and at points located at  $\pm 0.1$  times the standard deviation. The results of the SAP2000 analyses are presented in Table 3. Note that the response of interest to this example is the maximum bending moment in a bridge column. Two cases are considered: (1) the earthquake is applied in the longitudinal direction of the bridge and (2) the earthquake is applied in the transverse direction of the bridge. As mentioned earlier, the two cases are treated separately because the consideration of the earthquake orientation and the directional combination rules are beyond the scope of this study.

The results shown in Table 3 demonstrate that the foundation stiffness has negligible effect on the maximum moment response in the bridge columns. When the earthquake is applied in the transverse direction, no change is observed in the maximum bending moment of the columns. If the earthquake is applied in the longitudinal direction, a change of only 1.7% is observed when the change in the foundation stiffness is equal to 34% (2 times the standard deviations around the mean value). Changes of  $\pm 1$  standard deviation in the weight of the structure result in a change of less than 5% in the maximum bending moment. On the other hand, changes between 40% and 50% in the bending moments are observed when the spectral accelerations and the modulus of elasticity are changed by  $\pm 1$  standard deviation each. A change in the expected 75-year earthquake peak ground acceleration (PGA) by  $\pm 0.10$  standard deviation results in a change of about 13% in the columns' maximum bending moment. When the information summarized in Table 3 is used

in a first order Taylor series expansion, the response surface may be represented by an equation of the following form:

$$M_{\text{analysis transverse}} = -6,409 + 12,523 EQ + 0.002879 E + 3,790 C' + 9,735 b_W + 0 b_{K_s} \quad (1)$$

and

$$M_{\text{analysis long}} = -11,765 + 23,322 EQ + 0.00568 E + 6,900 C' + 17,090 b_W + 0 b_{K_s} \quad (1')$$

where

$M_{\text{analysis transverse}}$  and  $M_{\text{analysis long}}$  = the maximum moment response due to earthquakes in the transverse and longitudinal directions, respectively;

$EQ$  = the earthquake PGA;

$E$  = the modulus of elasticity;

$C'$  = the modeling variable of the spectral acceleration; and

$b_W$  and  $b_{K_s}$  = factors that multiply the weights,  $W$ , and the foundation stiffness,  $K_s$ , respectively.

The normalized factors,  $b_W$  and  $b_{K_s}$ , are used because each bridge element may have a different weight, and each foundation stiffness may be different. The normalized factors  $b_W$  and  $b_{K_s}$  in Equations 1 and 1' express how the final moment changes when all the individual weights and stiffnesses are varied by the same factor. When the mean values of the weights are used,  $b_W$  is set at 1.0. Similarly, when the mean values of  $K_s$  are used,  $b_{K_s}$  is set at 1.0. The model assumes that all the element weights are correlated such that a change of the weight of one element by a certain factor leads to the multiplication of all the other elements' weights by the same factor. A similar assumption is made for the foundation stiffnesses.

The results shown in Table 1 indicate that the design procedure proposed in NCHRP Project 12-49 [ATC and MCEER,

**TABLE 3 Results of parametric analysis close to mean values**

	<b>EQ Transverse</b> Moment about $x$ (kip-ft)	<b>EQ Longitudinal</b> Moment about $z$ (kip-ft)
Base case = analysis at mean values	3,879	6,900
75-yr $EQ + 0.1 \sigma_{EQ}$	4,054	7,251
75-yr $EQ - 0.1 \sigma_{EQ}$	3,560	6,331
$C' + \sigma_{C'}$	4,849	8,626
$C' - \sigma_{C'}$	2,954	5,176
$W + \sigma_W$	3,970	7,057
$W - \sigma_W$	3,786	6,734
$E + \sigma_E$	4,595	8,289
$E - \sigma_E$	3,006	5,153
$K_s + \sigma_{K_s}$	3,879	6,841
$K_s - \sigma_{K_s}$	3,879	6,960

2002) would dictate that the columns be designed to have maximum capacities  $M_{cap\ transverse} = M_{design\ transverse} = 1,606$  kip-ft (9,638.5 kip-ft/ $R_m$ ) where the 9,638.5 kip-ft is the moment obtained by analyzing the structure under the effect of the 2,500-year earthquake and the nominal values of the material properties and spectral accelerations, and  $R_{m\ nominal} = 6.0$  is the nominal response modification factor specified by NCHRP Project 12-49 for multicolumn bents. For the longitudinal direction, the required moment capacity is  $M_{cap\ long} = M_{design\ long} = 2,800$  kip-ft.

Failure occurs when the applied moment is higher than or equal to the design moment. The applied moment obtained from the SAP2000 analysis does not account for the nonlinear response that is normally represented using a response modification factor,  $R_m$ . In addition, to account for the modeling uncertainties, Ellingwood et al. (1980) have suggested that a modeling factor  $\lambda_{EQ}$  must also be included in the reliability analysis. This would lead to a failure function,  $Z$ , that can be represented by the equation

$$Z = M_{cap} - \lambda_{EQ} M_{analysis} / R_m \tag{2}$$

where  $M_{cap}$  is a random variable that, according to Nowak (1999), follows a lognormal distribution with a bias equal to 1.14 and a COV of 13%. The modeling factors  $\lambda_{EQ}$  and  $R_m$  are also random variables having the properties described in Table 2 above, and  $M_{analysis}$  is a function of the random variables  $EQ$ ,  $C'$ ,  $E$ ,  $W$ , and  $K_s$  (or, more precisely,  $b_w$  and  $b_{Ks}$ ). In the first iteration, the function that gives  $M_{analysis}$  is described in Equation 1 when the bridge is subjected to earthquakes in the transverse direction and in Equation 1' when the bridge is subjected to earthquakes in the longitudinal direction. Monte Carlo simulations using Equations 1 or 1' with Equation 2 produced reliability index values of  $\beta = 1.76$  for the transverse direction and  $\beta = 1.73$  for the longitudinal direction. Through the review of the various cases that fall within the

**TABLE 4 Results of parametric analysis close to most likely failure point**

$R_M = 6$	<i>EQ</i> Longitudinal	<i>EQ</i> Transverse
	Moment about <i>z</i> (kip-ft)	Moment about <i>x</i> (kip-ft)
Base case = most likely failure point	18,851	10,722
$EQ^* + 0.1\sigma_{EQ}$	19,352	10,986
$EQ^* - 0.1\sigma_{EQ}$	18,351	10,444
$C'^* + 0.2\sigma_{C'}$	19,714	11,216
$C'^* - 0.2\sigma_{C'}$	17,987	10,228
$W^* + 0.2\sigma_W$	18,948	10,772
$W^* - 0.2\sigma_W$	18,752	10,658
$E^* + 0.2\sigma_E$	19,566	11,097
$E^* - 0.2\sigma_E$	18,116	10,311
$K_s^* + 0.2\sigma_{K_s}$	18,814	10,722
$K_s^* - 0.2\sigma_{K_s}$	18,887	10,722

failure region, the Monte Carlo simulation also indicates that the most likely failure point occurs when  $EQ^* = 0.56g$ ,  $E^* = 1.36E_n$  ( $b_w^* = 1.05$ ) or  $W^* = W_n$ ,  $b_{C'}^* = 1.07$  and when  $b_{K_s}^* = 1.0$  or  $K_s^* = K_{sn}$  for the earthquake in the transverse direction. The maximum likely failure point occurs when  $EQ^* = 0.60g$ ,  $E^* = 1.39E_n$ ,  $W^* = 1.05W_n$ ,  $C'^* = 1.08$ , and  $K_s^* = K_{sn}$  when the earthquake is applied in the longitudinal direction. These most likely failure points are then used to perform a second iteration of the parametric analysis with results described in Table 4. These will in turn lead to new sets of response functions similar but with slightly different coefficients than those in Equations 1 and 1'.

The use of the results presented in Table 4 along with Equation 2 within a second and third reliability analysis leads to an updated reliability index of  $\beta = 1.76$  for the transverse direction and 1.74 for the longitudinal direction. The analysis is then repeated for various values of  $M_{cap}$  and is illustrated in Figure 4.

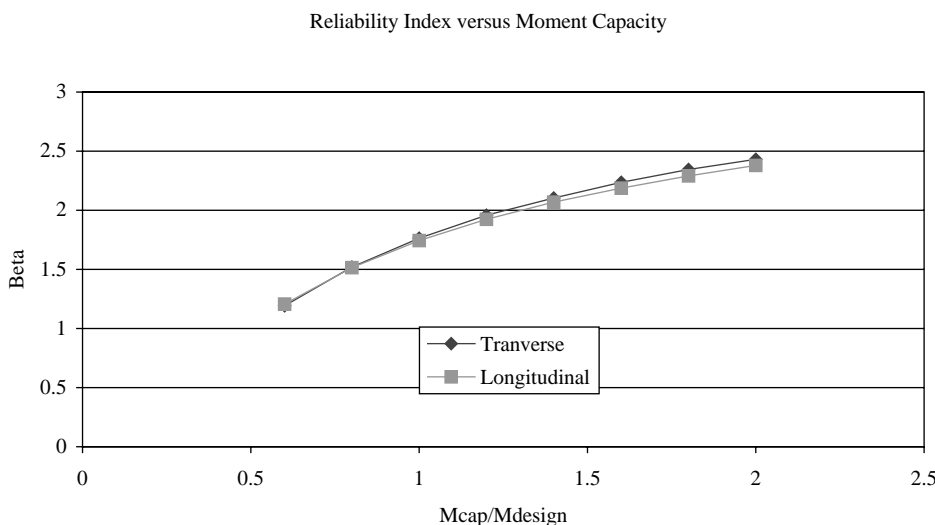


Figure 4. Variation of reliability index versus moment capacity.



The results of Figure 4 show that the reliability index values for the longitudinal and transverse directions produce a reliability index on the order of 1.75 if the proposed design guidelines of NCHRP Project 12-49 were to be adopted ( $M_{cap}/M_{design} = 1.0$ ) (ATC and MCEER, 2002). The reliability index would increase to close to 2.35 when the moment capacity of the columns is selected to have capacities equal to twice the recommended design capacities ( $M_{cap}/M_{design} = 2.0$ ). This increase in the reliability index could then be achieved by, for example, requiring the use of a response modification factor during the design process of  $R_m = 3.0$  rather than the  $R_m = 6.0$  that has been recommended by NCHRP Project 12-49. It is to be noted that because the analysis accounted for the nonlinear behavior of the members, the calculations executed herein correspond to the reliability of the bridge system. The reader is reminded that reliability index on the order of  $\beta = 3.5$  has been used as the basis for the design of bridge members under gravity loads. The presence of system redundancy would increase the system reliability index under gravity to  $\beta > 4.0$ . The lower reliability index values for earthquake loads is justified in the engineering community on the basis of an informal cost-benefit analysis whereby expert chief bridge engineers contend that there is general satisfaction with current design procedures given the excessive economic costs that would be associated with increasing the current reliability levels.

The results in Figure 4 show that the reliability index values are lower than those calculated in the main body of this report (*NCHRP Report 489*). This observed difference is due to the fact that the analysis performed herein is for the bridge columns while the analysis in the main body of the report was for the extension pile in the bridge foundation. The difference is primarily caused by the use of a lower response modification factor of  $R_m = 1.5$  for the extension pile as opposed to the  $R_m = 6.0$  used herein for bridge columns supported on multi-pile foundations.

To study the effect of the nominal value of the response modification factor  $R_m$  used for design on the reliability index, the reliability analysis of this five-span bridge is repeated for different values of the response modification factor and compared with the three-span bridge example analyzed in the main body of the report. The results for the case in which the earthquake is applied in the transverse direction are compared in Figure 5.

The results shown in Figure 5 demonstrate how the reliability index increases with the use of higher response modification factors and how the results obtained from this example based on an MDOF SAP2000 analysis of a multispan bridge founded on multi-pile foundation systems approach the results obtained in the body of the report for bridges on extension piles analyzed as SDOF systems. It is clear from the results that the major difference is due to the use of different response modification factors for different bridge members.

### 3. CONCLUSIONS

This appendix executed a reliability analysis for the seismic response of a five-span bridge with columns supported on multi-pile foundations and compared the results with those obtained in other parts of this report for bridges analyzed as SDOF systems. The results illustrate the following points:

1. The simplified analysis using an SDOF system yields results consistent with those obtained from the more advanced multimodal structural analysis.
2. RSM yields reasonable approximations to the reliability index for the seismic risk analysis of simple bridge configurations.
3. The use of a response modification factor of  $R_m = 6.0$  for bridge columns associated with the 2,500-year NEHRP spectrum produces system reliability index values on the order of 1.75. This is much lower than the member

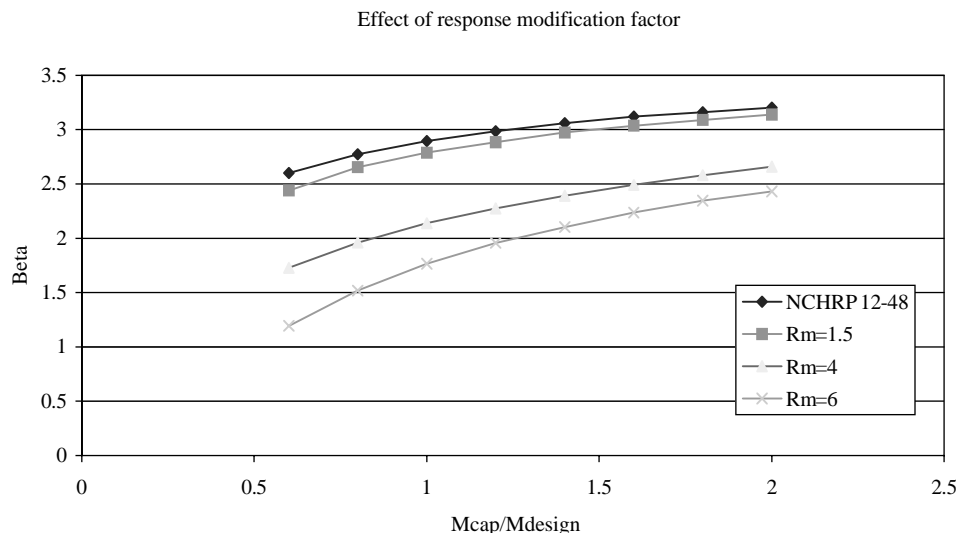


Figure 5. Effect of response modification factor on reliability index.

reliability equal to 3.5 used as the basis for calibrating the LRFD code for members under gravity load.

4. Large differences in the reliability indexes are observed for the different components of bridge subsystems subjected to seismic events. For example, the foundation systems produce much higher reliability index values than do the bridge columns. This difference has been intentionally built into the bridge seismic design process by the code writers (i.e., the NCHRP Project 12-49 research team) in order to account for the consequences of the failure of different members. However, the difference is rather large, producing a beta that varies from a value of about  $\beta = 3.0$  when a response modification factor equal to  $R_m = 1.0$  is used to about  $\beta = 1.75$  when a response modification factor of  $R_m = 6.0$  is used during the design process. It is noted that the use of a response modification factor of  $R_m = 1.0$  implies a design based on the elastic behavior of members while an  $R_m = 6.0$  accounts for the plastic behavior of bridge columns.
5. Although not common in the realm of seismic bridge engineering, the use of the reliability index as a measure of bridge safety for all types of extreme events will provide a consistent measure that will help in calibrating load combination factors as executed during the course of this study (see Chapter 3 of this report).

## REFERENCES FOR APPENDIX H

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