IMPLEMENTATION OF RISK-TARGETED GROUND MOTION APPROACH FOR SEISMIC DESIGN OF CONCRETE BRIDGES- Washington State Case Study

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Abstract

Design of bridges using risk-targeted ground motion (RTGM) ensures uniform seismic safety across diverse geological hazards, replacing traditional uniform hazard spectrums with a focus on a consistent, low probability of structural collapse. Adopted in the 2023 AASHTO Seismic Bridge Design Guide Specifications, enhancing reliability and optimizing costs. Unlike uniform hazard methods that only consider the likelihood of a certain ground motion, RTGM adjusts the design motion to ensure a consistent, uniform probability of collapse across different seismic regions, regardless of how fast the hazard increases. RTGM combines seismic hazard curves with structural fragility functions, often assuming a 5% conditional probability of collapse at the design intensity. This approach fits within performance-based seismic design (PBSD), allowing engineers to target specific damage states for different seismic demand levels. RTGM provides a more rational, consistent, and equitable level of safety for transportation infrastructure, often leading to more efficient designs in areas with high hazard variability. This paper focuses on the application of RTGM to seismic design of bridges and provides a seismic design comparison between the 2023 AASHTO adopted RTGM and the previous seismic design using the uniform hazard ground motions for a continuous concrete bridge with circular columns in high seismic regions. The variation in the design parameters is considerable because of the large variation in seismic hazard. The design comparison shows that in areas of high seismic regions using RTGM requires concrete columns with higher reinforcement ratios or larger diameters. This paper presents a side-by-side design comparison of a three-span highway bridge in Everett, Washington, designed under both the 2nd and 3rd Editions of the AASHTO Seismic Guide Specifications. The objective is to quantify the impact of the shift to RTGM on

seismic demand, displacement ductility requirements, and final column reinforcement design.

Keywords: Bridge, Seismic, RTGM, PBSD, Design Comparison

Introduction

Over the past decade, risk-targeted seismic design emerged as one of the most promising approaches for designing structures with controlled seismic risk and levels. Various risk-targeted design methods have been developed and applied to solve a wide range of design problems. Designing a structure to resist earthquakes by targeting an explicit failure risk has been a key research topic for many years.

The design ground motions used in these Guide Specifications are based on a risk-targeted approach, which differs from the uniform-hazard ground motions that were previously utilized.

The paper illustrates a risk-targeted method for the seismic design of bridges. In particular, the proposed procedure addresses the design problem for RC piers in multi-span bridges. The design parameters are the pier diameter, longitudinal and transverse reinforcement ratio, which are the most important parameters that control the performance of a bridge pier designed according to capacity design principles.

Uniform-Hazard Ground Motions

Uniform-hazard ground motions, which are derived on the basis of having an equal probability of being exceeded across the country, have been used for many years to support the seismic design of bridges, buildings, and other structures. For any given location, a hazard curve can be developed that expresses the probability of the spectral acceleration at a given period being exceeded over a defined time of exposure. This probability of exceedance can also be expressed in terms of a return period. For example, if the ground motion occurrence follows a Poisson process, a seven percent probability of exceedance in 75 years is approximately equivalent to having a 1000-year return period (McGuire, 2004). The most recently adopted AASHTO uniform-hazard design ground motions use a probability of exceedance of seven percent in 75 years. However, there is a disadvantage with this approach. The shape of the hazard curves for different locations may vary significantly, such that the choice of the design probability of exceedance for the ground motions (return period) affects the level of seismic ground motions at one location relative to another.

Risk-Targeted Ground Motions

It is important to understand the difference between seismic risk and hazard. Seismic hazard is given by the probability that an earthquake demand parameter, such as peak ground acceleration or spectral acceleration, at a specific site will be exceeded over a defined time of exposure. Seismic risk, on the other hand, relates to the likelihood that damage or loss will occur. Risk probabilities usually represent a level of damage to a structure that is expected to be exceeded over a defined time of exposure. Note that both are defined in terms of a probability of exceedance over a time window, but seismic risk evaluates damage or loss, whereas seismic hazard evaluates a metric of ground shaking intensity. Seismic risk convolves these seismic hazards with the structure's ability to resist that hazard without exhibiting or exceeding a prescribed damage level.

The desire to use seismic risk (the likelihood of undesired consequences from ground shaking, for example damage to structures), rather than hazard, has a long history, going back at least to 1948 when the U.S. Coast and Geodetic Survey produced a "Seismic Probability Map" which divided the contiguous United States into four zones (Roberts and Ulrich, 1949). A relevant feature of the map was qualitative estimation of earthquake-induced damage that could be expected regionally- ranging from "no damage" to "major damage" depending on the zone. The map did not attempt to provide quantitative estimates of the ground shaking itself. The map was adopted by the International Conference of Building Officials for inclusion in the Uniform Building Code (UBC), and as late as 1985, this type of qualitative "risk", not hazard, was the basis of the UBC seismic zonation. The implication of applying the provisions of the UBC was, if one followed the design provisions that corresponded to each zone of the map, damage of the type listed on the map would be mitigated. However, given the tools of the time, true probabilistic estimation of risk was not possible. As the field of seismology evolved, a probabilistic definition of seismic hazard was developed and adopted, but true risk-based seismic design was not yet possible. Today, the ability to couple the consequences of ground motion in a probabilistic manner to hazard, as described in the preceding paragraph, is possible. The risk-targeted approach thus brings seismic design back to consideration of the effect of the ground motions rather than focusing on hazard.

Risk-targeting of design ground motions is accomplished by jointly considering the seismic hazard over all return periods relative to the probability of exceeding a specific damage limit-state (e.g., incipient collapse, liquefaction) as a function of the ground shaking intensity (e.g., peak ground acceleration or spectral acceleration). This joint consideration of site hazard and geotechnical or structural damage is accomplished by numerical integration of two functions, one characterizing the seismic hazard and the other characterizing the likelihood of reaching a specific damage limit-state in what is termed a fragility function. In this "risk integral," expressed below, the hazard is denoted H(a) and the fragility Fr (a), where both are functions of the ground shaking intensity, denoted a. The risk calculates the probability of a damage limit-state being reached or exceeded over a given time of exposure (e.g., LS percent probability of collapse in 75 years), denoted here simply as R. The risk integral (e.g., Applied Technology Council, 1978; McGuire, 2004) is merely an application of the law of total probability from statistics. While the form of the risk integral expressed here is written in terms of the mathematical derivate of the hazard that is an equivalent form that instead takes the derivative of fragility.

$$R = \int_{0}^{\infty} Fr(a) \cdot \frac{dH(a)}{da} da$$

The implementation of RTGM extends the LRFD philosophy originally adopted by AASHTO in the LRFD Bridge Design Specifications to seismic design. There is a direct analogy between calibrating the load and resistance factors used in LRFD to obtain a target reliability index and adjusting the design ground motions to obtain a consistent probability of incipient collapse. The calibration process for LRFD includes consideration of the statistical variation of the load in the form of the mean, the coefficient of variation, and the mean-to-nominal ratio, which taken together describe the probability function for the loading. In a similar way, the statistical variation in the resistance is also determined. These are combined to calculate the probability of failure, expressed as the reliability index, and the combination of load and resistance factors that result in the target reliability index are determined.

In the development of risk-targeted ground motions, the seismic hazard curve (see Figure C.2-1) is analogous to the statistical distribution of the load, and the fragility function is similar to an expression of the distribution of resistance. Due to differences in the nature of seismic hazards and the manner in which they are expressed, the calculation process for determining the probability of failure differs from an LRFD calibration, but the result is the same. For risk-targeted ground motions, instead of calibrating a load factor and a resistance factor, the mapped design ground motion is adjusted to achieve the target risk value.

Similar to the manner in which the calibration of LRFD was referenced to the results of the previous non-LRFD design specifications, the risk target for developing the AASHTO design ground motions was determined based on existing practice in the Western United States using the seven percent in 75-year uniform-hazard motions. It is believed that the seismic performance of designs executed to these requirements is generally acceptable. The risk-targeted process was then used to achieve uniformity of risk across the nation by adjusting the design ground motions. In general, this required increases in the ground motions in the Central and Eastern United States It is important to note that use of the 2018 USGS national seismic hazard model introduced significant differences from previous AASHTO design ground motions.

A fragility function expresses the likelihood of reaching or exceeding a damage limit-state (e.g., spalling of the concrete in a column) given a measure of intensity (e.g., spectral acceleration). A fragility function can be developed for various purposes. For example, a fragility function relating overall bridge damage to peak ground or spectral accelerations may be used to estimate damage to existing structures; programs such as Shake Cast or Hazard typically use such functions. Alternatively, a fragility function may be used to quantify the damageability of not-yet-built structures, conditioned on design ground motion levels. The latter fragility function type is termed here as a notional fragility function and used for development of risk-targeted design ground motions.

Notional Fragility Function for Bridge Design

For the purposes of developing risk-targeted design ground motions, a notional fragility function focused on the expected performance of bridges designed to the current specification requirements was adopted. This is needed for several reasons:

- Currently, fragility functions for incipient bridge collapse do not exist for all types of bridges constructed in the U S. In fact, such relationships may be bridge topology-specific, and for the purposes of developing national risk-targeted ground motions, a single fragility function is preferred over specific fragility functions for individual bridges or types of bridges.
- Due to the calibration of the risk-targeted process, RTGM are relatively insensitive to the specific parameters of the notional fragility function used, as long as they are reasonably representative of bridge response (Luco, 2015).
- The most common bridge substructure element in use across the United States is the reinforced concrete column, and it is typically the intended primary energy-dissipating element and thus is deemed to drive the seismic performance of most new bridges.
- The notional fragility function adopted represents the likelihood that damage to reinforced concrete columns will lead to incipient collapse of a bridge, versus loss of support or failure of capacity-protected elements. Consequently, the risk targeting described herein is primarily directed at the new design of bridges.

It is important to note that while the resulting risk-targeted design ground motions are relatively insensitive to the choice of the fragility relationship used in the risk integral, the calculated value of risk will vary for different fragility functions. Therefore, the value indicated in these Guide Specifications should be viewed as an approximate value of risk for any particular bridge and will be influenced not only by what earthquake resisting system is utilized, but also by various decisions made during the design. The key advantage of the risk-targeted approach implemented in these Guide Specifications is the inclusion of the variation of hazard curve shape with location. Further improvement in the understanding of seismic risk as it relates to any particular bridge or component is expected in the future, but the first step of moving from a hazard to a risk basis for seismic design represents a significant improvement in seismic design philosophy. Similarly, although the fragility function used is based on structural response, the resulting RTGM can be used for geotechnical design. The geotechnical risk may vary from that calculated based on the structural notional fragility function, but the variation of hazard curve shape with location is still accounted for.

The conditional probability at the mapped ground motion is selected as five percent, which represents a high confidence of low probability of failure as described by McGuire (2004). When this value is coupled with the relatively large logarithmic standard deviation of 0.6, which accounts for such things as record-to-record variation of ground motion response, uncertainty in analytical damage predictability, and other sources of uncertainty, then the corresponding median is calculated to be approximately 2.7 times the mapped ground motion. A minimum typical ratio of the intended design point and the onset of significant degrading strength of a reinforced concrete column is approximately two. Given that a bridge will typically not collapse when one structural element reaches this threshold, the required median value of 2.7 is reasonable. The resulting notional fragility function is shown in Figure 1. The five percent probability at the design ground motion suggests that a large percentage of collapse occurrences will occur at demands above the design value, and that percentage can be determined by the risk-targeting calculation.

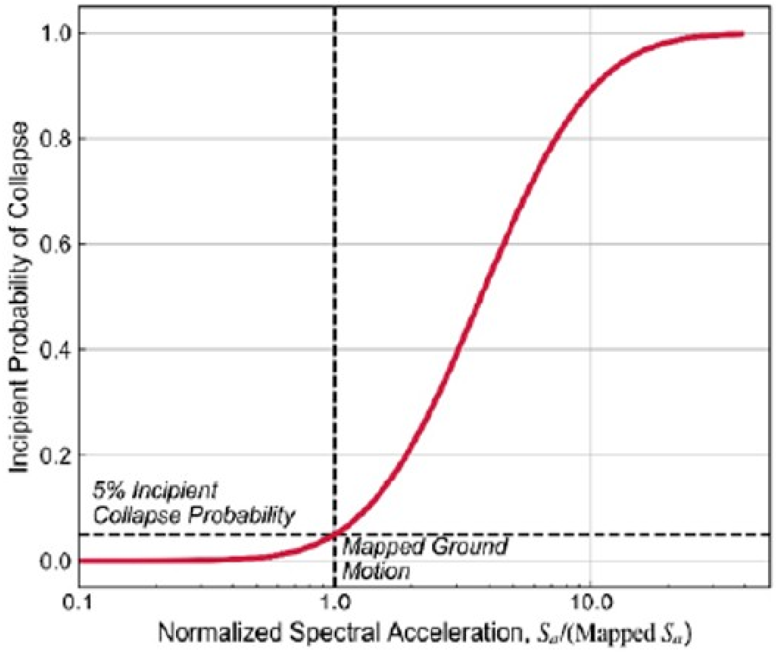

Figure 1 Notional Fragility Function, Conditioned on 5% Collapse Probability at the Mapped Ground Motion Level

The process of developing RTGM can be summarized as follows:

1. At each period, for each site class, beginning with an initial value for the design ground motion, calculate the risk using the hazard curve and notional fragility function (that depends on the design ground motion) through the risk integral equation.
2. Compare the calculated risk to the target value. Adjust the design ground motion level either upward or downward depending on whether the risk is higher or lower than the target.
3. Repeat steps 1 and 2 until the target risk value has been obtained. The resulting design ground motion is the risk-targeted ground motion for that period and site class.

Adjusting the design ground motion in Step 2 effectively shifts the fragility function left or right relative to the hazard curve. This can be seen by plotting the fragility function and hazard curve on the same graph, as shown in Figure 2. In this case, spectral displacement is used instead of acceleration, as the two have a one-to-one correspondence and for these purposes are equivalent.

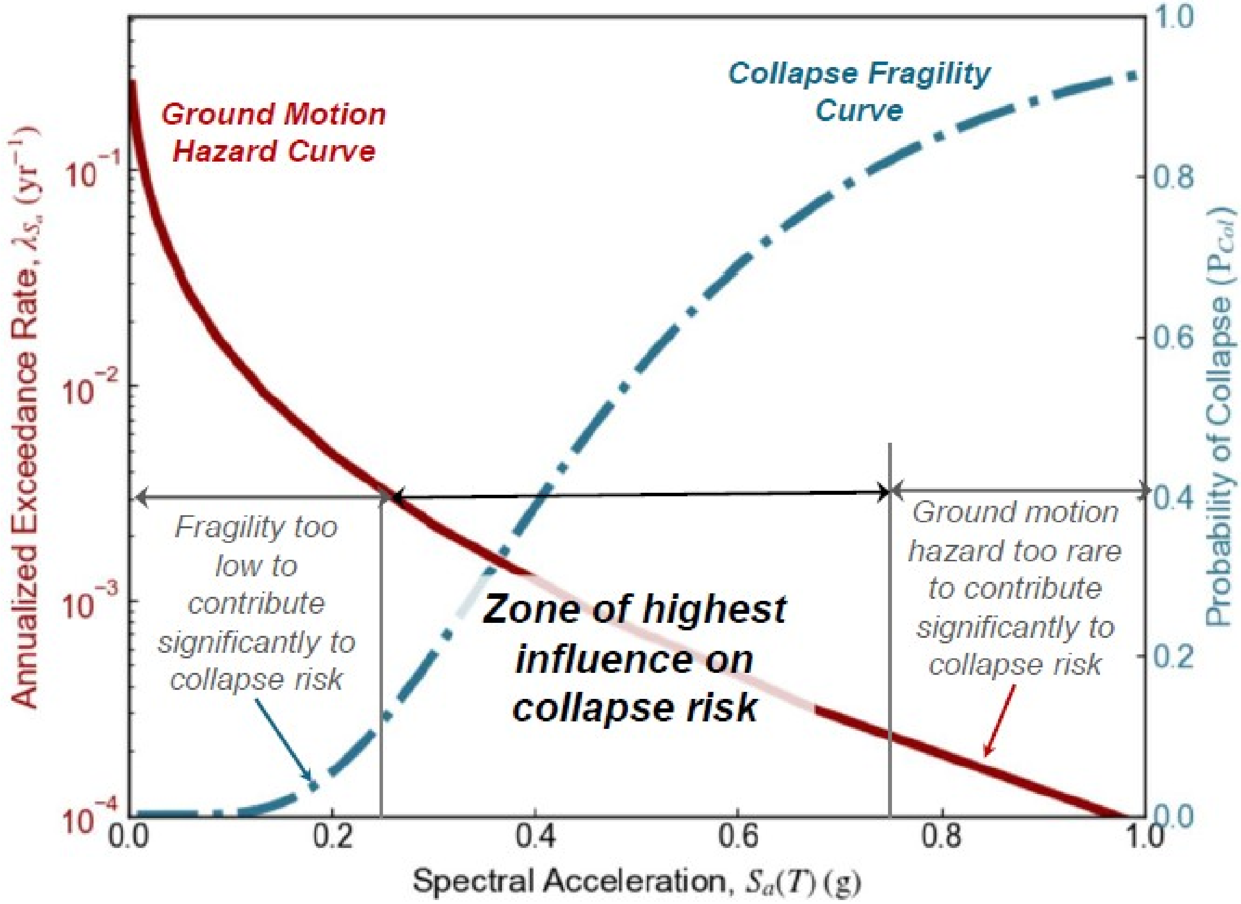

Figure 2 Illustration of Hazard, Fragility, and the Integration of the Two to Produce Risk.

Case Study Bridge Description

A three-span bridge located in Everett, WA was selected as the case study (Figure 3). The three spans measure 120 ft, 132 ft, and 120 ft, respectively, for a total back-to-back pavement seat length of 377 ft-6 in. The superstructure consists of six WSDOT WF83G precast prestressed concrete girders composite with a cast-in-place concrete deck.

The columns are connected integrally to the superstructure through a cap beam. Each column is 4 ft in diameter with the following reinforcement: 14 bundles of 2-#9 longitudinal bars ($\rho = 0.015$) and #5 hoops at 3.5 in. spacing for transverse reinforcement ($\rho s = 0.0082$). Concrete is 4,000 psi normal-weight, and reinforcing steel is ASTM A706 Grade 60. The foundations are oversized 7-ft diameter pile shafts. The abutments are seat-type on shaft foundations. The earthquake-resisting system (ERS) is classified as Type 1 per the AASHTO Seismic Guide Specifications, with the columns serving as the primary ductile elements.

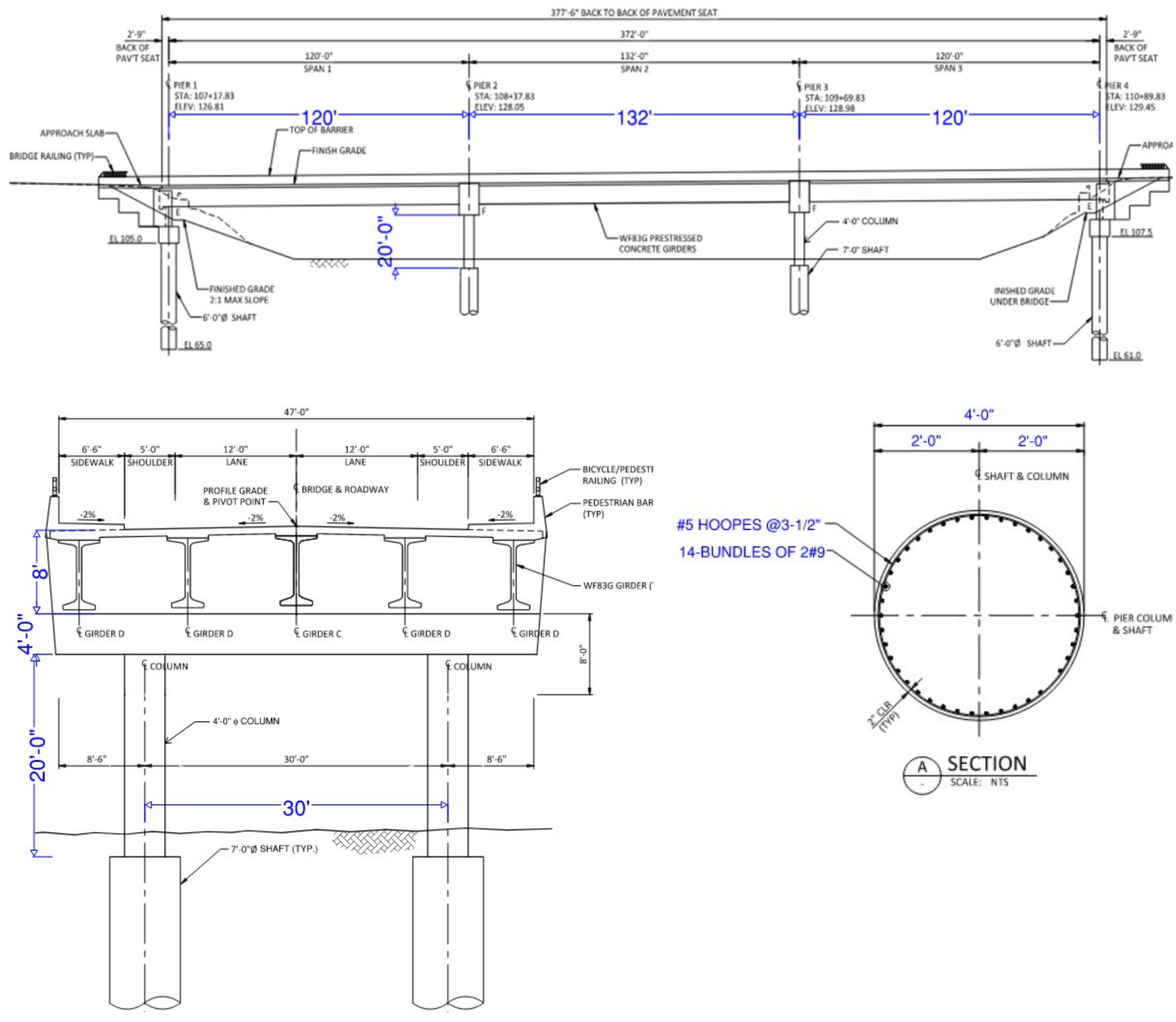

Figure 3 Bridge Detail

Response Spectrum

Response spectra for both the 2014 UHGM (7% PE in 75 years) and the 2023 RTGM (1.5% probability of collapse in 75 years) were obtained from the AASHTO web-based tools. The site is classified as Site Class D based on direct measurement of the shear wave velocity, vs = 900 ft/s. Initial geotechnical data from CPT soundings required analysis under three candidate spectra; however, a supplemental site exploration was performed to directly measure the shear wave velocity, which resolved the site classification and allowed analysis to proceed with a single spectrum.

Two spectral parameters differ between editions. Under the 3rd Edition, the SDS values are larger than those typically computed under the 2014 code in recent projects in the Pacific Northwest, leading to displacement amplification requirements for a broader class of structures. Table 1 presents the spectrum parameters for this site for both spectra.

Table 1 Spectrum Parameters

Parameter	**2014 UH**	**2023 RT**
(Equivalent) response spectrum acceleration coefficient at 1.0 s, S_{D1}	0.49	0.71
Ts	0.48 s	0.66
Characteristic ground motion period, T*	0.60	0.83
Seismic design category, SDC	C	D

Capacity Analysis

Pushover analyses were performed on the bridge bents using the initial column sizing. Foundations were modeled as fixed-base, and tributary axial loads (column dead load demands) were computed following the WSDOT construction sequence. In that sequence, the precast girders and CIP deck are placed prior to casting the pier concrete, making the dead load from those elements a simple-span contribution. After pier construction, barriers and sidewalks are placed and contribute as continuous beam loads.

The pushover analyses were performed for both the longitudinal and transverse directions. In both directions, the column is in double curvature with a clear height of 20 ft and an analytical plastic hinge length of 23.0 in. In the transverse direction, axial load variation due to the overturning moment is captured. The displacement capacities from the pushover analysis are summarized in Table 1.

Table 2 Pushover Analysis Results

Direction	**Longitudinal**	**Transverse**	
		$\mathbf{P_{min}}$	$\mathbf{P_{max}}$
$\mathbf{P_o}$ **(kip)**	1,450	820	2,080
$\mathbf{V_{po}}$ **(kip)**	530	467	583
$\mathbf{M_{po}}$ **(kip·ft)**	5,298	4,674	5,833
$\mathbf{\Delta_y}$ **(in)**	1.13	1.12	1.16
$\mathbf{\Delta_C}$ **(in)**	5.16	5.86	4.56

The column shear design was also verified as part of the capacity analysis. Per the AASHTO Seismic Guide Specifications, the shear capacity of columns depends on the displacement ductility demand. The displacement ductility at which the shear demand equals the shear capacity was determined as the ductility threshold and used subsequently to check against the actual seismic demand.

Seismic Demand Analysis

The bridge was analyzed using a three-dimensional finite element model in CSiBridge (BridgeModeler). The superstructure was modeled with beam and shell elements. Cap beams were modeled as frame elements with increased flexural and torsional stiffness to represent the stiff integral action of the integral connection. Columns were modeled as frame elements with cracked section stiffness, where the effective flexural stiffness was determined from the initial stiffness of the moment-curvature analysis (Icr/Ig = 0.473). The torsional stiffness was taken as 0.2 times the gross torsional stiffness. Abutments were modeled as rollers with no longitudinal or transverse contribution. The column bases were modeled as rigid (fixed-base).

Figure 1 shows the three-dimensional structural model and the first three mode shapes extracted from the modal analysis. Modes 1 (T = 1.26 s), 2 (T=0.83s), and 3 (0.82 s) are the torsional, transverse, and longitudinal responses, respectively. The high modal mass participation ratios confirm that three modes are sufficient to capture the dominant dynamic behavior of the bridge.

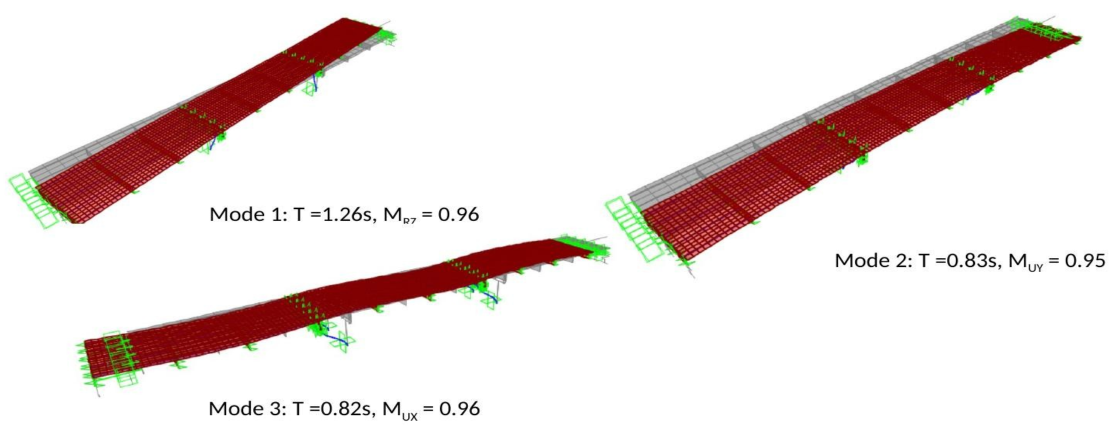

Table 3 The first three mode shapes

Design Check - Initial Column Design

Table 2 summarKhaleghiizes the seismic demand results for both the 2014 UHGM and the 2023 RTGM spectra against the applicable limits. The initial column design satisfies all demand limits under the 2nd Edition (2014 UHGM). However, under the 3rd Edition (2023 RTGM), the design fails in both the longitudinal and transverse directions, exceeding the displacement capacity and the displacement ductility limit required for shear capacity. The increase in seismic demands under the RTGM spectrum reflects the shift to SDC D and the larger spectral ordinates at the fundamental periods of the bridge.

Table 4 Design Check — Initial Column Reinforcement (#5 Hoops @ 3.5 in.)

Parameter	2014 UH	2023 RT	Limit	Status (2023 RT)
LONGITUDINAL				
Displacement (in)	3.9	5.4	5.16	**FAIL**
Ductility Demand (μD)	3.4	4.77	6[1] / 4.0[2]	**FAIL (exceeds μ_D limit for shear)**
P·Δ/Mp	0.05	0.07	≤ 0.25	PASS
TRANSVERSE				
Displacement (in)	3.8	5.1	4.6	**FAIL**
Ductility Demand (μD)	3.4	4.57	6[1] / 3.9[2]	**FAIL (exceeds μ_D limit for shear)**
P·Δ/Mp	0.07	0.10	≤ 0.25	PASS

[1] Member ductility limit per SGS Section 4.9. [2] Member ductility demand for shear capacity check. FAIL cells indicate non-conformance with 2023 RT limits.

Design Update and Revised Design Check

To satisfy the demands of the 3rd Edition AASHTO Seismic Guide Specifications, two remediation strategies were considered: (1) increasing the column diameter to reduce displacement demands, and (2) increasing the transverse reinforcement to improve ductility and shear capacity. The second option was selected as it is more economical and requires no change to the structural geometry.

The transverse reinforcement was upgraded from #5 hoops at 3.5 in. to #6 hoops at 3.5 in. (ρs = 0.0116). This modification increases both the displacement capacity and the shear capacity of the column. The updated design check is presented in Table 3.

Table 5 Design Check — Updated Column Reinforcement (6 Hoops @ 3.5 in.)

Parameter	2014 UH	2023 RT	Limit (Updated)	Status (2023 RT)
LONGITUDINAL				
Displacement (in)	3.9	5.4	6.26	PASS

Ductility Demand (μD)	3.4	4.64	6[1] / 5.3[2]	PASS
P·Δ/Mp	0.05	0.07	≤ 0.25	PASS
TRANSVERSE				
Displacement (in)	3.8	5.1	5.6	PASS
Ductility Demand (μD)	3.4	4.65	6[1] / 4.7[2]	PASS
P·Δ/Mp	0.07	0.10	≤ 0.25	PASS

[1] Member ductility limit per SGS Section 4.9. [2] Member ductility demand for shear capacity check with updated transverse reinforcement.

The updated design satisfies all displacement, ductility, and secondary moment checks for the 3rd Edition RTGM spectrum.

Design Comparison and Discussions

The longitudinal displacement demand increased from 3.9 in. to 5.4 in. (38%) and the transverse demand from 3.8 in. to 5.1 in. (34%), reflecting the larger RTGM spectral ordinates. Displacement ductility demands increased from 3.4 to approximately 4.7. An important distinction exists in the performance objectives. The bridge designed to the 2nd Edition under WSDOT Bridge Design Manual requirements is classified at the Recovery operational level, while the 3rd Edition design targets the Ordinary operational level.

Conclusions

This paper illustrated a risk-targeting design procedure for bridge piers. The procedure identifies the optimal values of the pier diameter and longitudinal reinforcement ratio that minimizes the resisting moment at the pier base while satisfying the mode of failure due to the exceedance of the pier displacement ductility capacity. This case study demonstrates several key findings from the transition to Risk-Targeted Ground Motions in AASHTO seismic bridge design:

1. The transition to RTGM results in a minor increase in analysis and geotechnical investigation costs.
2. Site characterization is critical. The initial CPT-based site data required evaluation under three spectra; direct shear-wave velocity measurements resolved the site class to D, allowing the analysis to proceed with a single, well-defined spectrum, improving design efficiency.

3. For this Everett site, the SDS under the 2023 code was larger than typical 2014 values, moving the bridge from SDC C to SDC D and increasing displacement demands by roughly 35%.
4. The 2014 spectrum achieves a Recovery operational level per the WSDOT BDM, whereas the 3rd Edition results in Ordinary. This may have significant implications for bridge inventory.

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