Changes in the New AASHTO Guide Specifications for Seismic Isolation Design

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Research Objectives

Two projects at the University at Buffalo under sponsorship of MCEER concentrated, respectively, on establishing new values of response modification factors for substructures of seismically isolated bridges, and on the study of the longevity and reliability of seismic isolation hardware. The latter culminated in the development of the concept of system property modification factors. This concept and the new values of response modification factors have been implemented in the new *AASHTO Guide Specifications for Seismic Isolation Design*.

In 1993, a project began at the University at Buffalo under the title "Longevity and Reliability of Sliding Seismic Isolation Systems" with the support of MCEER. The objectives of the project were then defined as the collection of laboratory and field data on the behavior of sliding bearings and the qualitative prediction of the long-term frictional properties of these bearings. In 1995, the author of this paper became involved in the development of the new *AASHTO Guide Specifications for Seismic Isolation Design* as a member of a task group of the T-3 Seismic Design Technical Committee of the AASHTO Bridge Committee. Specific challenges for the T-3 task group were the proposal of new response modification factors for bridge substructures and the justification thereof, and the development of a rational procedure for determining bounding values of isolator properties for analysis and design.

Based on the needs of the T-3 task group, the objectives of the research project were modified to include the development of a procedure for establishing bounding values of isolator properties. Moreover, a new project began in 1996 at the University at Buffalo with the support of MCEER to develop appropriate response modification factors for the substructures of seismically isolated bridges. These efforts culminated in the establishment of the concept of System Property Modification Factors, the development of revised values for response modification factors, and the inclusion of both in the new *AASHTO Guide Specifications*, which were published in 1999 (American Association of Highway and Transportation Officials, 1999).

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Related Highway Project Tasks

- *Development of Earthquake Protective Systems for Bridge Retrofit: Stability of Elastomeric Bearings,* ***I.G. Buckle****, University of Auckland, Energy Dissipation Systems,* ***M. Shinozuka****, University of Southern California, Comparative Assessment,* ***M. Feng****, University of California, Irvine*
- *Evaluation of Mechanical Property Changes for HDNR Bearings Over Time,* ***J. Kelly****, University of California, Berkeley*
- *Field Testing of a Seismically Isolated Bridge,* ***S. Chen*** *and* ***J. Mander****, University at Buffalo, and* ***B. Douglas****, University of Nevada-Reno*

Changes in the New AASHTO Guide Specifications for Seismic Isolation Design

The new specifications were developed by the T-3 task group during the period of 1995 to 1997 by considering the then current state-of-practice and the results of completed and ongoing research efforts. A number of changes in the new specifications over the predecessor specifications of 1991 are significant, either because they drastically change the analysis and design procedures or because they impose constraints that limit the application of some isolation systems.

Some of the changes are:

- The methods of analysis have been modified to include the effect of the flexibility of the substructure. The substructure increases the flexibility of the structural system and results in a damping ratio that is less than that of the isolation system (provided that there is no inelastic action in the substructure). The result is a net increase in the displacement of the structural system, which usually is related to an increase in the isolation system displacement. These phenomena have been convincingly demonstrated in NCEER-funded research (Constantinou et al., 1993, Tsopelas et al., 1994).

- The requirements for sufficient lateral restoring force have been changed so that the use of isolation systems with very low restoring force is disallowed in order to prevent the accumulation of large permanent displacements and to reduce the sensitivity of the displacement response to the details of the seismic input. Experimental results from another NCEER-funded project (Tsopelas and Constantinou, 1994) was the impetus for the implementation of this change.

- The response modification factors (R-factors) for the substructure of isolated bridges has been reduced so that, effectively, the substructure remains elastic. The T-3 task group endorsed a proposal by the author and reduced the R-factor on the basis of a small number of analytical results and engineering judgement. Research conducted in the meantime established the

Results of this research have been included in the *AASHTO Guide Specifications for Seismic Isolation Design*, published in 1999, where they represent the two major changes over the predecessor 1991 Specifications. The concept of system property modification factors is considered an innovation in the design of seismically isolated bridges and has been proposed for inclusion in the *Structural Engineers Association of California Blue Book* and the *NEHRP Recommended Provisions*, which apply for buildings. It is expected that this design concept will be both mandated and regularly used in the design of seismically isolated building and bridge structures.

necessity for lower R-factors on the basis of a comprehensive analysis, and verified the appropriateness of the selected values (Constantinou and Quarshie, 1998).

- A procedure for determining bounding values of the isolator properties for analysis and design has been included. This procedure is based on the determination of system property modification factors, or λ-factors, which account for the effects of aging, environment, contamination, history of loading and other conditions on the mechanical properties of isolators. The concept represents a drastic departure from previous practice and it is a bold procedure for considering the long-term behavior of the isolators rather than just their short-term performance in the laboratory. The concept, together with an extensive collection of data to support it, has been the result of a long-term MCEER-funded project (Constantinou et al., 1999).

Response Modification Factor

Response modification factors (R-factors) are used to calculate the design forces in the substructures of bridges from the elastic force demand. That is, the demand is calculated on the assumption of elastic substructure behavior and subsequently the design forces are established by dividing the elastic force demand by the R-factor.

The R-factor consists of two components. That is,

$$R = R_{\mu} \cdot R_{o} \qquad (1)$$

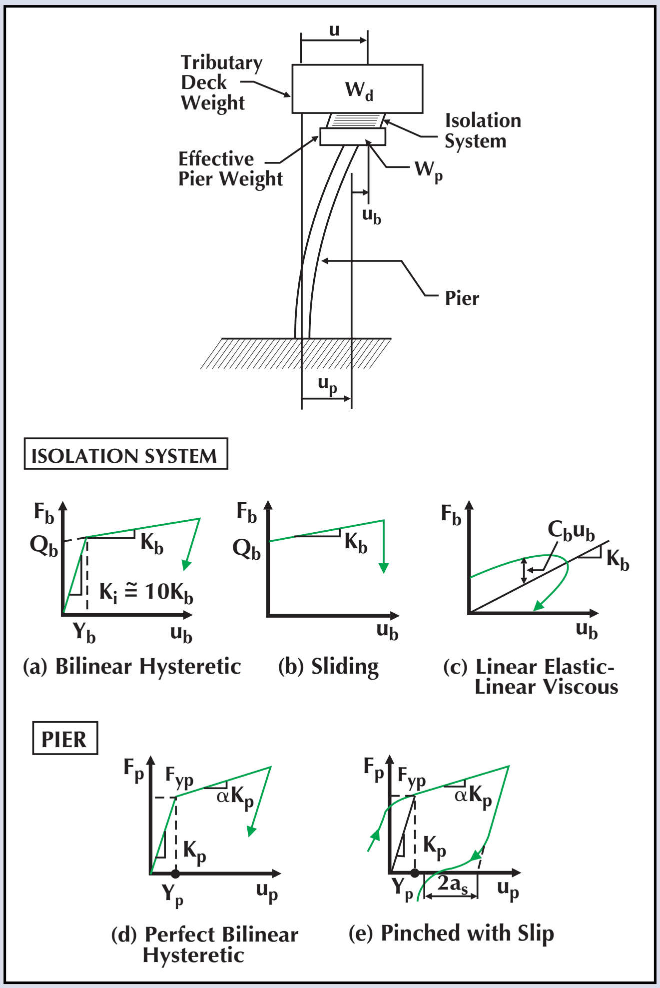
where R_{μ} is the ductility-based portion and R_{o} is the overstrength factor. The ductility-based portion is the result of inelastic action in the system. The overstrength factor is the result of reserve strength that exists between the design force and the actual yield strength. Single column substructures of bridges have no overstrength (that is, R_{o} = 1.0), whereas multiple column bent substructures have overstrength which typically is assumed to correspond to R_{o} = 1.67.

The ductility-based portion of the R-factor has been presumed to be related to the ability of the substructure to undergo inelastic action. Accordingly, the original 1991 *AASHTO Guide Specifications for Seismic Isolation Design* specified R-factors that were identical to those specified for the substructures of conventional, non-isolated bridges. The assumption was thus made that the inelastic demand in the substructures of seismically isolated and non-isolated bridges would be the same if the two were designed for the same R-factor.

This presumption was incorrect. The demand in the substructure of seismically isolated bridges is strongly dependent on the relation between the strength of the isolation system and the strength of the substructure. It is apparent that the strength of the substructure should be higher than that of the isolation system, or otherwise the isolation system becomes totally ineffective. This principle has been convincingly demonstrated in a series of simple examples by Constantinou and Quarshie (1998), who also performed a systematic study for establishing appropriate values for the R-factor.

Results of Analysis on R-factors for Seismically Isolated Bridges

Figure 1 shows a simple deck-isolation system-substructure model used in the study of Constantinou and Quarshie (1998). A variety of behaviors for the isolation system and substructure are shown in Figure 1, and a range of parameters were considered in the dynamic analysis of this system. Analysis was performed as follows:



■ Figure 1. Analyzed System and Illustration of Utilized Force-Displacement Relations for Isolation System and Pier

- For a particular combination of parameters characterizing the system, analysis was performed assuming elastic substructure behavior and utilizing the simplified analysis procedures of AASHTO.

- The strength of the substructure was then established as the calculated elastic force demand divided by the R-factor. The latter is now just the ductility-based portion since the system lacks redundancy.

- The system was then analyzed and its nonlinear response history was calculated for 20 earthquake motions which were appropriately scaled to represent the applicable response spectrum.

- The analysis results were used to calculate, among other quantities, the average displacement ductility ratio in the substructure which provided the most useful information in establishing appropriate values of the R-factor.

Table 1 presents a summary of selected results from these analyses. To appreciate the level of inelastic action in the substructure of the isolated bridges, analyses were also performed for non-isolated bridges. Table 2 presents a sample of results obtained from such analyses, which is appropriate to compare with the sample of results in Table 1. Such a comparison reveals that the ductility ratio in the substructure of isolated bridges is more, actually much more, than that of non-isolated bridges when both are designed for the same R-factor.

■ Table 1. Average Substructure Displacement Ductility Ratio of Isolated Bridges

System	R_μ=1.0	R_μ=1.5
Bilinear Hysteretic Pier, A = 0.4, Soil Type II, Bilinear Hysteretic Isolation System δ = 0.06	1.2-1.8	2.4-4.7
Bilinear Hysteretic Pier, A = 0.4, Soil Type II, Bilinear Hysteretic Isolation System δ = 0.10	1.3-2.1	2.8-6.5
Bilinear Hysteretic Pier, A = 0.4, Soil Type II, Linear Elastic/Viscous Isolation System ξ = 0.2	1.4-1.6	3.0-4.0
Bilinear Hysteretic Pier, A = 0.4, Soil Type II, Linear Elastic/Viscous Isolation System ξ = 0.3	1.4-1.5	3.0-4.2
Bilinear Hysteretic Pier, A = 0.4, Soil Type III, Bilinear Hysteretic Isolation System δ = 0.06	0.9-1.4	2.2-4.3
Bilinear Hysteretic Pier, A = 0.4, Soil Type III, Bilinear Hysteretic Isolation System δ = 0.10	0.9-1.5	1.9-4.9
Bilinear Hysteretic Pier, A = 0.4, Soil Type III, Linear Elastic/Viscous Isolation System ξ = 0.2	0.9-1.6	1.7-3.9
Bilinear Hysteretic Pier, A = 0.4, Soil Type III, Linear Elastic/Viscous Isolation System ξ = 0.3	0.9-1.3	2.2-4.0
Pinched Hysteretic Pier, A = 0.4, Soil Type II, Bilinear Hysteretic Isolation System δ = 0.06	1.5-2.1	2.8-5.4
Pinched Hysteretic Pier, A = 0.4, Soil Type II, Bilinear Hysteretic Isolation System δ = 0.10	1.5-2.4	3.1-6.7

■ Table 2. Average Substructure Displacement Ductility Ratio of Non-Isolated Bridges

System	R_μ=1.0	R_μ=2.0	R_μ=3.0
Bilinear Hysteretic Pier A = 0.4, Soil Type II	0.9	1.9	3.5
Bilinear Hysteretic Pier A = 0.4, Soil Type III*	1.4	3.1	5.2
Pinched Hysteretic Pier A = 0.4, Soil Type II	0.9	2.9	4.6

*** Conservative values**

"It is expected that the system property modification factors concept will be both mandated and regularly used in the design of seismically isolated building and bridge structures."

On the basis of such comparisons, and additional results on the sensitivity of the substructure inelastic response of isolated bridges, it was concluded that the ductility-based portion of the substructures of isolated bridges should be less than or equal to 1.5. Moreover, analyses of the overstrength in isolated bridges have shown that, in general, the overstrength is slightly higher than in non-isolated bridges. Finally, values of the R-factor for isolated bridges have been established and are presented in Table 3. Nearly identical values (1.5 instead of 1.67) have been included in the new *AASHTO Guide Specifications for Seismic Isolation Design*.

Table 3. Proposed Values of R-factor for Substructures of Isolated Bridges

Substructure	R_μ	R_o	R
Wall-Type Pier (Strong Direction)	1.0	1.67	1.67
Wall-Type Pier (Weak Direction, Designed as a Column)	1.5	1.0	1.5
Single Columns	1.5	1.0	1.5
Multiple Column Bent	1.5	1.67	2.5

System Property Modification Factors

The properties of seismic isolation bearings vary due to the effects of wear, aging, temperature, history of loading, and so on. The exact state of the bearings at the time of seismic excitation cannot be known. However, it is possible to establish maximum and minimum probable values of important properties (i.e., characteristic strength and post-yielding stiffness) within the lifetime of the structure. The analysis can then be conducted twice using the bounding values of properties. In general, the maximum force and displacement responses will be obtained in these analyses.

In principle, the probable maximum and minimum property values could be established on the basis of statistical analysis of the variability of the properties and the likelihood of occurrence of relevant events, including that of the considered seismic excitation. This is an admittedly very difficult problem. However, it is relatively easier to assess the effect of a particular phenomenon on the properties of a selected type of bearing, either by testing (e.g., effect of temperature on friction coefficient in sliding bearings) or by a combination of testing, rational analysis and engineering judgement (e.g., effect of aging). This leads to the establishment of system property modification factors, that is, factors which quantify the effect of a particular phenomenon on the nominal properties of an isolation bearing, or system in general.

Consider that a nominal value of a property of an isolation system is known. It could be that this value is assumed (on the basis of experience from previous testing) during the analysis and design phase of the project or it is determined in the prototype bearing testing. Typically, this nominal value applies for specific conditions, such as fresh bearing conditions, temperature of 20°C and the relevant conditions of vertical load, frequency or velocity and strain or displacement. Let this value be P_n.

The minimum and maximum values of this property, P_{max} and P_{min} respectively, are defined as the product of the nominal value and a series of System Property Modification Factors, or λ-factors as follows:

$$P_{max} = \lambda_{max} \cdot P_n \quad (2)$$

$$P_{min} = \lambda_{min} \cdot P_n \quad (3)$$

where

$$\lambda_{max} = \lambda_{max,\,1} \cdot \lambda_{max,\,2} \cdot \lambda_{max,\,3} \cdots \quad (4)$$

$$\lambda_{min} = \lambda_{min,\,1} \cdot \lambda_{min,\,2} \cdot \lambda_{min,\,3} \cdots \quad (5)$$

Each of the $\lambda_{max,i}$, $i = 1,2 \cdots$ factors is larger than or equal to unity, whereas each of the $\lambda_{min,\,i}$, $i = 1,2 \cdots$ is less or equal to unity. Moreover, each of the λ-factors is associated with a different aspect of the isolation system, such as wear, contamination, aging, history of loading, temperature, and so on.

As an example, consider the effect of temperature on the friction coefficient of a sliding bearing. The range of temperature over the lifetime of the structure is first established for the particular site or general geographic area of the project. This range need not be one of the extreme (lowest and highest) temperatures. Rather, it could be a representative range determined by the responsible professional (more appropriately, this range could be included in the applicable specifications). Say this range of temperature is -10°C to 50°C. Testing is then performed at the two temperatures and the λ-factors are established as the ratio of the coefficient of friction at the tested temperature to the coefficient of friction at the reference temperature (say 20°C). Factor $\lambda_{min,\,t}$ will be based on the data for the highest temperature (50°C), whereas $\lambda_{max,\,t}$ will be based on the data for the lowest temperature (-10°C).

As another example, consider the effect of wear on the friction coefficient. On the basis of the geometric characteristics of the bridge (span, girder depth, etc.), average vehicle crossing rate and lifetime of the structure, the cumulative travel is determined. Test data are then utilized to establish the λ-factors for wear (or travel). Typically, $\lambda_{max,\,tr}$ is the ratio of the coefficients of friction determined in high velocity testing following to and prior to a sustained test at the appropriate velocity (~1mm/s) for a total movement equal to the calculated cumulative travel. The $\lambda_{min,\,tr}$ is determined in a similar manner but for a total movement less than the calculated cumulative travel for which the coefficient of friction attains its least value.

The system property modification factors are associated with different aspects of the isolation system and combined on the basis of (4) and (5). While each one of these factors describes the range of effect of a particular aspect, their multiplication results in a combined factor of which the value may be very conservative. That is, the probability that several events (such as lowest temperature, maximum travel, maximum corrosion, etc.) occur simultaneously with the design-basis earthquake is very small.

It is necessary that some adjustment of the system property modification factors is applied to reflect the desired degree of conservatism. This adjustment should be based on a statistical analysis of the property variations with time, the probability of occurrence of joint events and the significance of the structure. It is also desirable to apply this adjustment with the simplest possible procedure.

Such a procedure is based on system property adjustment factors, a, such that the adjusted value of the λ-factor is given by

$$\text{adjusted } \lambda_{max} = 1 + (\lambda_{max} - 1) \cdot a \qquad (6)$$

$$\text{adjusted } \lambda_{min} = 1 + (1 - \lambda_{min}) \cdot a \qquad (7)$$

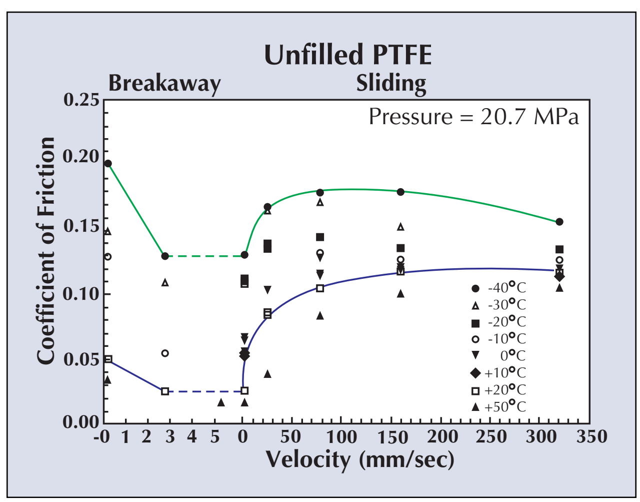
That is, the property adjustment factor is multiplied by the amount by which the λ-factor differs from unity and the result is added to unity to yield the adjusted λ-factor. It is evident that the adjustment factor can take values in the range of 0 to 1. The value a = 0 results in an adjusted λ-factor of unity (that is, variations in properties are disregarded – least conservative approach). The value a = 1 results in no adjustment (that is, the maximum variations are considered to occur simultaneously – most conservative approach).

The following system property adjustment factors have been proposed by the author and included in the new *AASHTO Guide Specifications*:

1	for critical bridges
0.75	for essential bridges
0.66	for all other bridges

These values are based on engineering judgement and a desire to employ the most conservative design approach for critical bridges. It is expected that as experience develops over the years of observation of the performance of seismically isolated bridges and other structures, and more data are collected on the variations of properties, more refined values of system property adjustment factors could be established.

Values of λ-factors have been established for sliding and elastomeric isolation systems on the basis of a long-term study which included a comprehensive review and analysis of available data, extensive testing, application of principles of solid mechanics and use of engineering judgement (Constantinou et al., 1999). There are too many



■ Figure 2. Friction of Unfilled PTFE-Polished Stainless Steel Interfaces at Various Temperatures as Function of Sliding Velocity

■ Table 4. System Property Modification Factor for Effects of Temperature ($\lambda_{max,t}$) on the Coefficient of Friction of Sliding Bearings

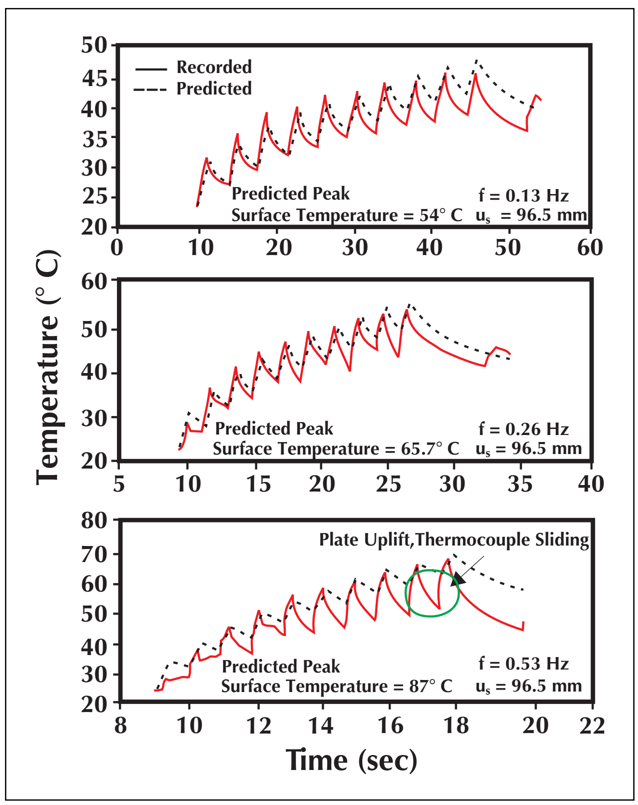
Temperature (°C)	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
20	1.0	1.0	N/A
0	1.1	1.3	N/A
-10	1.2	1.5	N/A
-30	1.5	3.0	N/A
-40	1.7	N/A	N/A
-50	2.0	N/A	N/A

factors to describe each one and the physical phenomena responsible for the effects. It is sufficient to present herein some representative results on one of the effects and the related λ-factors.

Low temperature causes an increase in the friction of PTFE-stainless steel interfaces used in sliding bearings and in the stiffness and characteristic strength of elastomeric bearings. The effect in the case of elastomers is time-dependent, that is, the increase in stiffness is greater with increasing time of exposure at a particular low temperature. For sliding interfaces, the effect of low temperature is highly dependent on the speed of sliding motion since frictional heating can cause substantial increases in temperature following very small travel.

While testing of seismic isolation bearings at low temperature is a relatively straight forward exercise (albeit not an easy one), the interpretation of the results and the establishment of λ-factors requires an understanding of the frictional heating problem. Figure 2 presents a sample of experimental results on the frictional properties of unfilled PTFE-highly polished stainless steel interfaces for a range of velocities of sliding, and temperature at the start of the experiment. The substantial effect of frictional heating, as made evident in the figure with the increased velocity, is apparent. On the basis of these and other results, which were generated in very time-consuming experiments, the λ-factors of Table 4 were developed (Constantinou et al., 1999) and incorporated in the new *AASHTO Guide Specifications*.

To assess the effect of frictional heating, an analytic solution was derived to predict the temperature rise at the sliding interface and at some depth below. Carefully planned experiments (including the use of extremely fine thermocouple wires) were also conducted to obtain reliable measurements of histories of temperature rise



■ Figure 3. Recorded and Predicted Histories of Temperature at Depth of 1.5 mm Below the Sliding Interface in Large Amplitude Tests for verification of the theory. Figure 3 presents a comparison of measured and predicted histories of temperature during testing of a sliding bearing.

Conclusions

Research supported by MCEER resulted in the establishment of new response modification factors for the substructures of seismically isolated bridges and in the development of a new concept in the analysis seismically isolated structures.

Both developments have been incorporated in the new *AASHTO Guide Specifications for Seismic Isolation Design*, which were published in 1999.

Of particular interest in this work is the development of the concept of system property modification factors and the substantial multi-year effort to establish values for these factors on the basis of testing, rational analysis and engineering judgement. Despite this effort, there are a significant number of problems that could not be adequately addressed in this research. Most important of these problems is that of the prediction of the aging characteristics of seismic isolation hardware, which requires a substantial, multidisciplinary basic research effort to adequately address.

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