Aging Considerations in the Development of Time-Dependent Seismic Fragility Curves

Jayadipta Ghosh, S.M.ASCE¹; and Jamie E. Padgett, A.M.ASCE²

Abstract: This paper presents the formulation of a time-dependent seismic fragility format for bridges, as well as new insights into the potential effects of aging and deterioration on seismic vulnerability traditionally neglected in fragility modeling, including joint impacts of multiple component deterioration not investigated to date. The study evaluates the impact of lifetime exposure to chlorides from deicing salts on the seismic performance of multispans continuous highway bridges, considering corrosion of reinforced concrete columns and steel bridge bearings. The components’ degradation and their influence on seismic response are illustrated through three-dimensional nonlinear dynamic analysis. A full probabilistic analysis accounting for variation in bridge, ground motion, and corrosion parameters is conducted to develop time-dependent seismic fragility curves. These fragility curves indicate the evolving potential for component and system damage under seismic loading considering time-dependent corrosion-induced deterioration. The results indicate that while corrosion may actually decrease the seismic vulnerability of some components, most critical components suffer an increase in vulnerability. Quadratic models depicting the change in lognormal seismic fragility parameters are proposed to capture the time-dependent effect of aging on the fragility of the bridge system. Overall, the seismic vulnerability significantly increases throughout the lifetime of the representative bridge geometry, with a 32% shift in the median value of complete damage fragility near the end of the bridge’s life.

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Introduction

Across the United States, hundreds of thousands of existing bridges are located in seismic zones (FHWA 2009). Regional risk assessment of this infrastructure has relied upon significant advances in the development of tools known as bridge fragility curves (Basoz and Mander 1999; Basoz and Kiremidjian 1999; Shinozuka et al. 2000; Mackie and Stojadinovic 2001; Nielson and DesRoches 2007a; Straub and Der Kiureghian 2008), which offer statements of the probability bridge failure conditioned upon intensity of ground motion. The integration of these conditional reliability models into regional seismic risk assessments provides an opportunity to screen seismically vulnerable bridges for retrofit, project anticipated damage and losses, or support postevent inspection. However, according to the American Society of Civil Engineers, over half of the 599,766 bridges in the United States are approaching the end of their design life and nearly a quarter need significant retrofit or replacement to eliminate deficiencies (ASCE 2009). The aging and deterioration of bridges manifest itself in a number of ways, such as spalling of reinforced concrete (RC) members, buildup of debris leading to corrosion of steel bearings, and corrosion of steel reinforcement in RC columns among others. These components comprise the primary resisting system of bridges under seismic loading thus having a potentially significant impact on seismic fragility estimates, which currently neglect these aging parameters and emphasize the as-built or pristine condition of bridges. Moreover, the time-dependent nature of bridge deterioration phenomena suggests that seismic fragility models of bridge components or systems should also reflect this evolving vulnerability in order to increase the reliability of their application in risk assessment of aging bridge inventories.

Despite the potential effect of deterioration across a large population of aged bridges on seismic performance, there has been a lack of historic consideration of the joint effect of seismic and aging threats. Recent notable work by Choe et al. (2008, 2009) has highlighted the potential reduction in capacity and increase in fragility of a typical single-bent bridge in California considering nonlinear static analyses in the derivation of individual component fragility models (namely, RC columns in marine splash zone). This work illustrated the potential importance of capturing the effects of aging on seismic fragility and identifying the crucial material and corrosion parameters that most significantly affect the bridge reliability. However, for other bridge types, such as multiple span steel or concrete girders bridges, past studies have illustrated the importance of capturing a number of vulnerable components in the fragility assessment of the bridge system (Nielson and DesRoches 2007b). Therefore, further research is required to evaluate the effect of aging on system response and fragility, considering not only the vulnerability of multiple components but also their simultaneous aging. Furthermore, the anticipated dynamic behavior of these systems under various time-dependent deterioration mechanisms has yet to be

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characterized to support the development of time-dependent seismic fragility curves.

A potential form of environmental degradation of bridges is chloride induced corrosion of bridge components such as RC columns and steel bridge bearings (Enright and Frangopol 1998; Stewart and Rosowsky 1998; Montemor et al. 2002; Hoeke et al. 2009). Corrosion deterioration of bridges is predominantly found to occur at locations with close proximity to the sea coast and in regions where deicing salts are applied for snow and ice removal. The study presented herein focuses on investigating the effects of corrosion due to chloride-laden deicing salts because of (1) their extensive use in bridges across the United States (Broomfield 1997) and (2) the identification of chloride induced corrosion from deicing salts as one of the most severe forms of corrosion which causes significantly higher degradation than chlorides in a marine environment (Stewart and Rosowsky 1998). The corrosion of critical components considered in this study includes corrosion of the steel reinforcement in RC columns and the deterioration of steel bridge bearings commonly found in existing inventories of steel girder bridges in seismically vulnerable regions such as the Central and Southeastern United States (CSUS). While bridge columns are potentially subjected to traffic spray of chloride-laden water under the bridge (Weyers et al. 1994), steel bridge bearings are affected by the leaking of chloride-laden water at deck joints (Silano and Brinckerhoff 1993).

This paper offers the formulation of a time-dependent seismic fragility format as well as new insights into the potential effects of aging and deterioration on seismic vulnerability traditionally neglected in fragility modeling, including joint impacts of bearing and column deterioration not investigated to date. The degradation mechanisms associated with corrosive chloride attack considered in this study include reduction in strength of bridge columns due to decrease in diameter of the corroded reinforcement, increased bearing coefficient of friction due to corrosion debris accumulation, and failure of the bridge bearing system due to corrosion of anchor bolts and keeper plates. The impact of these multiple component deterioration mechanisms is assessed within the framework of bridge system fragility estimation for a typical multispans continuous (MSC) steel girder bridge throughout its service life. Analytical models for corroded RC columns and steel bridge bearings are introduced into the probabilistic seismic performance assessment, which assesses system reliability considering the contribution of multiple correlated component failures (e.g., columns, bearings, and abutments). Finite-element modeling and nonlinear dynamic analysis of the typical bridge with its deteriorated components are conducted to provide new understanding of the effect of aging on seismic response at the individual component and bridge system level. A sampling-based analytical fragility analysis is then conducted considering uncertainty in hazard, bridge, and deterioration parameters to yield a comparison of the anticipated seismic performance at different points in time along the bridge’s service life. New time-dependent fragility models of the MSC steel girder bridge are formulated that capture the decrease in anticipated seismic performance as the bridge ages and suffers continued exposure to the elements. The discussion of results offers insights on the potential joint effects of deterioration of multiple components on lifetime bridge vulnerability under seismic loads. In addition, opportunities for future work are discussed, such as accounting for additional deterioration mechanisms, conducting field-condition updating of fragility estimates, or developing time-dependent fragility curves conditioned upon multiple parameters.

Case Study Bridge and Base Finite-Element Model

To demonstrate the methodology for developing time-dependent fragility curves and provide insight on the impacts of aging of multiple components on seismic vulnerability, a sample MSC steel girder bridge is used as a case study in this paper. The bridge is representative of the median dimensions among all MSC steel girder bridges found in the CSUS bridge inventory based on past statistical analyses (Nielson 2005). The MSC steel girder bridge is adopted for the case study because of the prevalence of this bridge class in regions of potential seismic hazard (i.e., attributing 13.2% of CSUS bridges in seismic zones according to Nielson 2005), typical design details including multiple components that are susceptible to corrosion, and concern regarding initial bridge vulnerability even prior to considering aging. Past comparative studies on classes of bridges, which are particularly common in the Central United States, indicate that MSC steel girder bridges are among the most vulnerable bridges (Nielson and DesRoches 2007b). This can be attributed to the inadequate seismic detailing of the columns having approximately 1% longitudinal reinforcement ratio along with widely placed transverse ties, using the vulnerable high type steel fixed and rocker bearings, short seat widths, and inadequately reinforced pile caps. Consequently, large inertial deck loads result in considerably high demands on the underreinforced columns, expansion bearings, and abutments during seismic events.

Bridge Geometry

The typical MSC steel bridge configuration used in this study is that identified by Nielson (2005), as shown in Fig. 1, illustrating the continuity of the steel girders over the interior bents. Both the end spans and the middle span of this three span bridge are 22.30 m long and 10.3 m wide consisting of five steel girders. Each bent consists of three circular columns having 645-mm² nominal cross-sectional area, reinforced with 12 No. 29 longitudinal bars (metric size), and No. 13 (metric size) transverse stirrups spaced at 300 mm. The bridge uses high type steel fixed bearings beneath each girder over the bent beam and high type steel expansion (rocker) bearings at the abutments. These bearings are placed on masonry plates and attached to the bridge pier and abutments using anchor bolts. Besides being highly prone to corrosion deterioration, the nonductile nature of these bearings makes them highly susceptible to seismic damage (Mander et al. 1996).

Finite-Element Modeling of Pristine Bridge Components

A three-dimensional finite-element model for the chosen bridge configuration is developed using the finite-element platform OpenSees (Mazzoni et al. 2009) following the recommendations by Nielson and DesRoches (2007a). An overview of the finite-element model for the nondegraded bridge is presented herein for completion prior to describing the influence of corrosion on the bridge model. For the superstructure modeling, the composite actions of the steel girders and bridge deck are taken into account and modeled with linear elastic beam-column elements since damage is not expected in the superstructure. Analytical modeling of the bridge bearings relies heavily on the experimental and analytical suggestions following the reversed cyclic loading tests on steel bridge bearings by Mander et al. (1996). Subsequently, bi-

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Corroded Reinforcement

\[ F_{y,corr} = f(u_{corr}) \]
\[ F_y = f(u) \]

where:
- \( D_c \) = diffusion coefficient;
- \( C \) = chloride ion concentration;
- \( x \) = depth of concrete from the surface; and
- \( t \) = time in years.

Assuming that the chloride ion concentration near the concrete surface is constant as typically assumed for corrosion due to deicing salts (Hoffman and Weyers 1996; Vu and Stewart 2000), the corrosion initiation time is therefore taken as (Thoft-Christensen et al. 1996)

\[ \frac{\partial C(x,t)}{\partial t} = -D_c \frac{\partial^2 C(x,t)}{\partial x^2} \]  

where \( D_c \) = diffusion coefficient; \( C \) = chloride ion concentration; \( x \) = depth of concrete from the surface; and \( t \) = time in years. Assuming that the chloride ion concentration near the concrete surface is constant as typically assumed for corrosion due to deicing salts (Hoffman and Weyers 1996; Vu and Stewart 2000), the corrosion initiation time is therefore taken as (Thoft-Christensen et al. 1996)
solves due to continued chloride ingress, corrosion initiates and to the simulated data for corrosion initiation time. This distribution dependent parameter with potential time dependence and Stewart 2000 considered to be constant on average along the service life of the exposed to deicing salts that can vary considerably for different bridges depending on bridge location and environmental exposure condition. The probabilistic models for the lognormally distributed parameters describing the corrosion initiation of the RC columns adopted for the present study are given in Table 1. These lognormally distributed random variables are identified based on in-field corrosion studies of existing bridge components in the United States exposed to deicing salts (Whiting et al. 1990; Weyers et al. 1994; Enright and Frangopol 1998). The distribution for the corrosion initiation time is assessed through Monte Carlo simulation having a sample size of 50,000. A lognormal distribution with mean 8.85 years and standard deviation of 4.5 years is found to be a good fit by the initial area of reinforcement, \(A_0\), which is the area of reinforcing steel at time \(t=0\). This figure illustrates the reduction in steel cross-sectional area over time and the increase in variability or uncertainty about that estimate of reinforcement area due to the corrosion rate model accounted for by adopting a probabilistic model for the corrosion rate rather than a single deterministic value, as shown in Table 1. Due to the potential influence of the corrosion rate model adopted, alternate models can be incorporated through the same approach presented in this paper for bridge-specific analysis. On the basis of the estimated distribution for corrosion initiation time presented above and anticipated corrosion rate, the area of reinforcing steel is probabilistically assessed as a function of time. Fig. 2 shows the resulting time-dependent area reduction ratio, which is the area of reinforcing steel at time \(t\), \(A(t)\), normalized by the initial area of reinforcement, \(A_0\). This figure illustrates the reduction in steel cross-sectional area over time and the increase in variability or uncertainty about that estimate of reinforcement area due to the combined effect of the variability of initial reinforcement diameter, rate of corrosion, and corrosion initiation time. The loss of area of steel due to corrosion of the RC columns is correspondingly modeled as a reduction in longitudinal reinforcing bar cross-sectional area in the fiber section model as compared to the pristine columns in the finite-element model.

### Deterioration of Steel Bridge Bearings due to Corrosion

Degraded seismic performance of the bridge bearings results primarily due to the corrosion debris accumulation resulting in "frozen" or "locked" bearings, as well as the corrosion of anchor bolts and keeper plates used in bearing assemblies. Typically, such anchor bolts are used in both fixed and expansion (rocker) bearings, while the keeper plates are typical details in the expansion bearing assemblies. As identified by Mander et al. (1996) bearing anchor bolts often form a "weak link" in the chain of force transmission from the superstructure to the substructure during seismic events. Corrosion of these same elements may potentially result in a shift in performance during seismic loading. The primary reasons for the corrosion of the bearing assembly are the leaking of chloride-laden water from the deicing salts through the deck

### Table 1. Descriptors of Lognormal Random Variables Affecting the Corrosion Deterioration of RC Columns

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Unit</th>
<th>Mean</th>
<th>COVa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover depth (x)</td>
<td>cm</td>
<td>3.81</td>
<td>0.20</td>
</tr>
<tr>
<td>Diffusion coefficient ((D_r))</td>
<td>cm²/year</td>
<td>1.29</td>
<td>0.10</td>
</tr>
<tr>
<td>Surface chloride concentration ((C_{0r})) wt % concreteb</td>
<td></td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Critical chloride concentration ((C_{cr})) wt % concreteb</td>
<td></td>
<td>0.040</td>
<td>0.10</td>
</tr>
<tr>
<td>Rate of corrosion ((r_{corr}))</td>
<td>mm/year</td>
<td>0.127</td>
<td>0.3</td>
</tr>
</tbody>
</table>

aCOV=coefficient of variation.
bwt % concrete=percent by weight of concrete.

\[
T_i = \frac{x^2}{4D_r} \left[ \text{erf}^{-1} \left( \frac{C_0 - C_{cr}}{C_0} \right) \right]^{-2}
\]

where \(T_i\)=corrosion initiation time; \(C_0\)=equilibrium chloride concentration at the concrete surface; \(C_{cr}\)=critical chloride concentration that causes dissolution of the protective passive film around the reinforcement and initiates corrosion; and \(\text{erf}^{-1}\) is a Gaussian error function that can be mathematically represented as (Edwards 2006)

\[
\text{erf}(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-t^2} dt
\]

The corrosion initiation time depends on a number of parameters that can vary considerably for different bridges depending on bridge location and environmental exposure condition. The probabilistic models for the lognormally distributed parameters describing the corrosion initiation of the RC columns adopted for the present study are given in Table 1. These lognormally distributed random variables are identified based on in-field corrosion studies of existing bridge components in the United States exposed to deicing salts (Whiting et al. 1990; Weyers et al. 1994; Enright and Frangopol 1998). The distribution for the corrosion initiation time is assessed through Monte Carlo simulation having a sample size of 50,000. A lognormal distribution with mean 8.85 years and standard deviation of 4.5 years is found to be a good fit to the simulated data for corrosion initiation time. This distribution will subsequently be used as a key input for probabilistic modeling of rebar corrosion for bridge columns.

Once the protective passive film around the reinforcement dissolves due to continued chloride ingress, corrosion initiates and the time-dependent loss of reinforcement cross-sectional area, \(A(t)\), can be expressed as (Thoht-Christensen et al. 1996; Enright and Frangopol 1998)

\[
A(t) = \begin{cases} 
\frac{nD^2t^2}{4} & \text{for } t \leq T_i \\
\frac{n[D(t)]^2}{4} & \text{for } T_i < t < T_i + D/r_{corr} \\
0 & \text{for } t \geq T_i + D/r_{corr}
\end{cases}
\]

where \(n\)=number of reinforcement bars; \(D_t\)=initial diameter of steel reinforcement; \(t\)=elapsed time in years; \(r_{corr}\)=rate of corrosion; and \(D(t)\)=reduction diameter at the end of \((t-T_i)\) years, which can be represented as

\[
D(t) = D_t - r_{corr} \cdot (t - T_i)
\]

It is acknowledged that the corrosion rate is an environmentally dependent parameter with potential time dependence (Vu and Stewart 2000). However, the rate of corrosion in this study is considered to be constant on average along the service life of the bridge. The primary reason behind this assumption is the traditional lack of explicit data for time-dependent corrosion rate modeling, which also prompted previous researchers (Frangopol et al. 1997; Val et al. 2000; Akgül and Frangopol 2004; Liu 2005) to conduct bridge reliability studies using models with average corrosion rates. In this study, uncertainty in the corrosion rate is accounted for by adopting a probabilistic model for the corrosion rate rather than a single deterministic value, as shown in Table 1. Due to the potential influence of the corrosion rate model adopted, alternate models can be incorporated through the same approach presented in this paper for bridge-specific analysis. On the basis of the estimated distribution for corrosion initiation time presented above and anticipated corrosion rate, the area of reinforcing steel is probabilistically assessed as a function of time. Fig. 2 shows the resulting time-dependent area reduction ratio, which is the area of reinforcing steel at time \(t\), \(A(t)\), normalized by the initial area of reinforcement, \(A_0\). This figure illustrates the reduction in steel cross-sectional area over time and the increase in variability or uncertainty about that estimate of reinforcement area due to the combined effect of the variability of initial reinforcement diameter, rate of corrosion, and corrosion initiation time. The loss of area of steel due to corrosion of the RC columns is correspondingly modeled as a reduction in longitudinal reinforcing bar cross-sectional area in the fiber section model as compared to the pristine columns in the finite-element model.
joint (Silano and Brinckerhoff 1993) and the traffic spray scenarios which may further expose these components to airborne chlorides resulting from the passage of vehicles beneath the bridge through chloride-laden water (Enright and Frangopol 1998). Recent in-field examples of severe anchor bolt corrosion exist in the literature for typical highway bridges in the state of Georgia (Hoeke et al. 2009). In regions where use of deicing salts on bridge decks is more prevalent, the severity of anchor bolt corrosion may be even more critical. Additionally, accumulation of excessive corrosion products, dirt, and debris may potentially result in frozen or locked bearings since it may restrict translational and rotational movements due to an increased coefficient of friction (Silano and Brinckerhoff 1993).

Anchor bolt corrosion leads to reduced ultimate lateral strength of the bearing assembly as explained in the following paragraphs. Fig. 3 shows the arrangement and distribution of forces for a typical fixed bearing along the longitudinal direction. This free-body diagram is used to derive the ultimate lateral strength for the fixed bearing in the longitudinal direction, which changes over time due to corrosion. From the equilibrium of horizontal forces, the ultimate lateral strength of the bearing can be obtained as

\[ F_{ult} = \alpha S + \mu V \]  

and from the equilibrium of vertical forces we have

\[ V = N + \alpha B \]  

Also, from the equilibrium of moments about the center of the concrete pedestal we obtain

\[ F_{ult}h = V \left( \frac{w_f - \alpha}{2} \right) \]  

where \( \alpha \) = number of anchor bolts; \( S \) = shear force on one anchor bolt; \( \mu \) = coefficient of friction between masonry plate and bedding material; \( V \) = compression force on the concrete pedestal due to rocking; \( N \) = axial load on the bearing; \( B \) = bond strength of the swedged anchor bolt in the concrete pedestal; \( h \) = height of the bearing from the concrete pedestal to the sole plate-rocker interface; \( w_f \) = width of masonry plate in the longitudinal direction; and \( a \) = depth of pedestal concrete stress block expressed as \( a = V/0.85f_w w_f \), where \( w_f \) is the width of masonry plate in the transverse direction and \( f_c \) is the concrete compressive strength.

The bond strength of the anchor bolt may be estimated as (Mander et al. 1996)

\[ B = b_s (\pi d_b) l \]  

where \( b_s = k b_r \) = bond stress that is assumed to act uniformly on the anchor bolt surface with diameter \( d_b \) over the embedment length \( l_e \); \( b_r \) = average bond stress over the length of the anchor bolt; and \( k \) = modification or judgment factor often imposed to account for reduced bearing capacity of the anchor bolts and adverse effects of cyclic loading. From the above equations, the ultimate lateral strength, \( F_{ult} \), of the bearing may be expressed as

\[ \frac{F_{ult}}{N} = \frac{0.5w_f}{h} \left( 1 + \frac{\alpha B}{N} \right) - \frac{N}{0.85f_w w_f} \left( 1 + \frac{\alpha B}{N} \right)^2 \]  

While a similar relationship for estimating the ultimate strength of fixed bearing along the transverse direction can be found in available literature (Mander et al. 1996), it should be noted that the calculated strengths along both directions correspond to that of the pristine bearings. Along the service life of the bridge due to corrosion, the cross-sectional area of the bolt decreases and consequently leads to reduced ultimate lateral strengths for the deteriorated bearings. The corrosion parameters previously defined in Table 1 are also assumed to affect the corrosion initiation time and subsequent area loss of steel in the concrete embedded anchor bolts. This reduction in cross-sectional area impacts the bond strength, \( B \), of the embedded bolt as indicated in Eq. (8). Uncertainty in the corrosion parameters via reduction in bond strength is then propagated through the assessment of ultimate lateral strength of the fixed bearing along the longitudinal and transverse directions. This is shown for the longitudinal direction in Fig. 4. Consequently, the time-evolving probability distribution for ultimate lateral strength is accounted for in the finite-element modeling for longitudinal and transverse fixed bearing responses.

For the case of expansion (rocker) bearings, the motion in the longitudinal direction is primarily rocking, where the ultimate lateral strength is dependent on the coefficient of rocking friction of the bearing. As per suggestions by Mander et al. (1996), the coefficient of rocking friction varies from 0.04 for clean well worn rocker bearings to 0.12 for badly corroded bearings to take into account the “locking” effect as mentioned earlier. In the absence of any further data to support time-dependent modeling of friction increase from corrosion product buildup, a linearly varying coef-
cient of friction is assumed for this study starting with 0.04 for the pristine bridge and 0.12 for a 100-year-old bridge near the end of its service life.

The most interesting case observed in this study is the effect of aging on the transverse model of the expansion bearing response. The transverse motion of the expansion bearing initially consists of a sliding frictional component. Once the horizontal frictional force exceeds the frictional resistance of the sole plate-rocker interface, the sole plate slides on the rocker until the rocker bearing strikes the keeper plate provided to prevent excess transverse motion. With additional horizontal loading, the keeper plate bends significantly and fails by tearing of the fillet weld securing the plate (Mander et al. 1996). The free-body diagram in Fig. 5 shows the distribution of forces for this phenomenon.

As it can be seen, the force $P$ with which the rocker strikes the keeper plate gets transmitted in the form of shear forces $S_1$ and $S_2$ through the anchor bolts. For intact bearings with no deterioration, the shear strength of the 25-mm diameter anchor bolt is found to be sufficient to transmit the forces, with the failure of the bearing governed solely by the tearing failure of the keeper plate. However, with corrosion deterioration, there is significant decrease in the shear strength of the anchor bolts, such that they are no longer capable to transmit the forces when the rocker strikes the keeper plate. This refers to the phase where the failure of the bearing assembly is determined by the shear failure of the anchor bolts rather than the failure of the keeper plate.

In addition to the corrosion of the concrete embedded anchor bolts, the transverse failure mechanism and ultimate strength of the expansion bearings as previously described are also a function of the chloride exposed steel keeper plates. The corrosion degradation of the keeper plate is modeled similar to past studies on corrosion of steel bridge girders. The corrosion of these exposed steel elements is assumed to follow an empirical model following the form of a power law (Komp 1987)

$$y(t) = P_t^Q$$

where $y(t)$=average corrosion penetration in micrometers; $t$=time in years; and $P$ and $Q$=parameters determined from regression analysis of field experimental data. Parameters $P$ and $Q$ are assumed to follow a truncated correlated bivariate lognormal distribution and are determined based on field tests by Albrecht and Naeemi (1984). The parameter means, coefficients of variation, and correlation coefficient used in this study are obtained from regression analysis of corrosion penetration field tests on carbon steel girders due to deicing salt corrosion. Estimated values of mean and coefficient of variation are found to be 53.5 μm and 0.20 for $P$ and 0.6 and 0.4 for $Q$, with a correlation coefficient of −0.55 between the parameters. Steel bearing keeper plates, typically made of carbon steel, are assumed to undergo similar corrosion penetration.

Fig. 6 shows the degradation in the shear strength of the anchor bolt over time due to corrosion. Also plotted is the maximum shear force that can be transmitted to each anchor bolt when the expansion bearing strikes the keeper plate in the transverse direction. Beyond this force level in the anchor bolt, the keeper plate yields and does not transmit further loads to the anchor bolt. Along with the mean values presented in the figure, the full probabilistic analysis conducted herein considers variation about both quantities. As illustrated in the figure, before approximately 70 years, the failure of the bearing assembly is dictated by failure of the keeper plate since the maximum force in the bolt is much less than its shear strength capacity. However, beyond 70 years, the capacity of the anchor bolt decreases below the level of force transmitted. Hence, the failure of the bearing is governed by shear failure of the anchor bolt. This phenomenon is captured by adjusting the ultimate strength of the keeper plate assembly in the expansion bearing model, depending on the point in time along the bridge’s service life.

**Scope of Deterioration Modeling and Future Opportunities**

It is acknowledged that other possible bridge deterioration mechanism and permutations of component aging are feasible, which are outside of the scope of the present study and modeling. Emphasis is placed upon corrosion of steel reinforcement in RC columns, corrosion of keeper plates and anchor bolts in bearing assemblies, and increased coefficient of friction for the steel bearings. These mechanisms are among the most severe phenomena associated with corrosion of aging bridges exposed to deicing salts identified in a series of past studies (Silano and Brinckerhoff 1993; Pantazopoulou et al. 2001; Lindquist 2008; Li et al. 2009). Furthermore these phenomena affect the components of the MSC steel bridge class identified as the most significant and vulnerable components under seismic loads and hence deemed a priority when incorporating effects of aging phenomena within the seismic reliability analysis. However, it is noted that the corrosion of reinforcing steel within the concrete columns may lead to second-
ary effects such as cracking and spalling of concrete and loss of bond strength, among others, in addition to the loss of reinforcement cross-sectional area considered herein. These additional phenomena highlight the opportunity for future study, although preliminary analysis conducted as a part of this research indicates that some of these degradation effects can be neglected owing to their limited impact on the bridge fragility.

Corrosion of steel girders in the bridge superstructure has, for example, received significant attention in the literature with regard to corrosion process modeling and reliability assessment of bridges under live loading (Kayser and Nowak 1989; Czarnecki and Nowak 2008). However, a preliminary sensitivity study conducted by the writers revealed the negligible impact of this process on the bridge’s seismic vulnerability. Even in the event that additional girder or superstructure components are added to the system fragility definition, the demands placed on a 100-year corroded bridge girder under seismic loading are not anticipated to exceed the yield capacity of the component and hence remain elastic under seismic loading and negligible in the fragility assessment. Additionally, concrete cover loss due to spalling and bond strength loss are not modeled in this study for the RC columns or other components. A preliminary sensitivity analysis revealed that the effect of column concrete cover loss on bridge vulnerability is insignificant, resulting in only a 2–6% shift in the PGA value for the corroded bridge column with complete cover loss when compared to the corroded bridge column with intact cover across the different damage states. Moreover, explicit time-dependent modeling of concrete spalling leading to cover loss is not a trivial task and is expected to introduce additional uncertainty in the fragility estimation. Traditionally, studies on concrete cover loss due to spalling have assumed uniform loss of cover along the whole length of the column which is seldom the case observed in the field. The results of this sensitivity study, however, do not diminish the potential influence that early aged cracking could have on fragility due to its likely impact on corrosion initiation time, not explored herein. Furthermore, loss of bond strength is acknowledged to be a potential additional mechanism requiring future research. Previous studies have shown that for RC members with insufficient splice lengths and no confinement, the loss of bond strength is significant (Fang et al. 2004; Aquino and Hawkins 2007), whereas for members with reinforcement confinement, the loss of bond strength due to corrosion is not substantial (Fang et al. 2004). The modestly confined columns considered in the case study bridge, typical of preseismic design detailing, fall in between the two categories noted. The limited availability of experimental data for model validation and capacity estimation of bond strength that affects the seismic response of corroded columns, including poorly confined columns, further highlights the need of future experimental and analytical research. Additionally, some other forms of deterioration of the RC members, such as concrete strength degradation due to freeze-thaw cycles and chemical attacks along with the aforementioned factors, provide opportunities for future study.

Impact of Corrosion-Induced Component Deterioration on Seismic Response and Capacity of Bridge Components

Before conducting a full probabilistic analysis of the impact of corrosion on the fragility of the representative MSC bridge, a sample deterministic simulation is presented to illustrate the influence of time-dependent aging on the seismic response of the MSC bridge. For the specific bridge type and geometry considered, the first two fundamental modes are longitudinal and transverse modes, with respective periods of 0.34 and 0.25 s. The dynamic response is illustrated through nonlinear time history analysis of the bridge with median values for all variable parameters using a ground motion from the synthetic suite by Rix and Fernandez (2004). This motion has peak ground acceleration (PGA) of 0.5 g and duration of 29 s. In addition to the dynamic response of the structure in both the longitudinal and transverse directions, the influence of corrosion on the load resisting capacity is also presented. For brevity, comparisons are made between the pristine bridge at time zero and the bridge at 50 years into its service life. The following sections illustrate the seismic demands due to individual consideration of deterioration mechanisms of the components as well as the demands due joint consideration of both column and bearing deterioration mechanisms.

Impact of Corrosion on the Seismic Response of RC Columns

The effect of time-dependent corrosion of the RC column is first assessed. Due to corrosion and subsequent area loss of reinforcing steel, the load carrying capacity and yield curvature of the RC columns undergo a significant reduction. This phenomenon is illustrated in Fig. 7(a), which shows a 16.6% reduction in yield curvature and a 21% reduction in the yield moment of a 50-year-old corroded column as compared to that of a pristine column. Subsequently, when the bridge is subjected to the sample ground motion, the demands placed on the corroded RC column increase relative to the pristine column. This finding is illustrated in Fig. 7(b). The seismic demand placed on the columns is quantified by the curvature ductility demand ratio, which can be expressed as

$$\mu_\phi = \frac{k_{max}}{k_{yield}}$$

where $k_{max}$ corresponds to the maximum curvature demanded on the column throughout the seismic loading and $k_{yield}$ = curvature in the column which causes first yield of the outermost reinforcing bar. While the peak curvature ductility demand of 3.3 for the pristine bridge already indicates significant damage in the form of cracking and spalling, after 50 years of exposure to deicing salts the bridge subjected to the same motion suffers peak curvature ductility demands of 5.4, which signifies a more severe damage in the form of column reinforcement buckling (Hwang et al. 2001; Buckle 2006).

An increased seismic demand on the corroded RC columns is found to correspond to a negligible increase in demands on certain components such as expansion bearings and abutments, which only show approximately 3 and 1% increases, respectively, in peak displacement in the corroded bridge relative to the pristine bridge. In some components, however, there is a reduction in the peak seismic demands when column corrosion is modeled. For instance, compared to the pristine bridge, there is an approximately 13% reduction in the peak longitudinal displacement of the fixed bearings in the 50-year-old bridge. This reduction in bearing demands is attributed to the concentration of damage in the corroded columns as compared to the pristine bridge.
Impact of Corrosion on the Seismic Response of Steel Bridge Bearings

Corrosion deterioration and subsequent strength reduction of high type steel fixed and expansion bridge bearing assemblies are addressed in this section. Recalling the deterioration model presented in the previous section, corrosion of the steel bearings results in reduced stiffness and reduced ultimate strength in the fixed bearing assemblies. While this is also true for expansion bearings along the transverse direction, increased coefficient of friction along the longitudinal direction however results in increased expansion bearing stiffness. Analogous to the response of columns in the previous section, considering only bearing degradation results in an increase in the peak displacement of the fixed bearing assembly by 16 and 11% in the longitudinal and transverse directions, respectively. As expected, the reduced post yield stiffness and ultimate strength due to corrosion shift the hysteretic characteristics of the bearing and result in larger peak deformations under seismic loading. Fig. 8 shows the comparative response of longitudinal loading response of the fixed bearings for the pristine bridge and 50-year-old bridge.

For expansion bearings, the increase in coefficient of friction due to debris accumulation increases the yield force by 19% and reduces deformation of the expansion bearings in the longitudinal direction by 21%. Additionally, in the transverse direction, the reduced ultimate strength of the bearing assembly results in an 18% increase in peak deformation for the 50-year-old bridge bearing as compared to the pristine bearing.

Joint Consideration of the Impact of Corrosion on the Seismic Response of the MSC Steel Girder Bridge

To reflect field conditions for corroded bridges throughout their lifetime, the joint occurrence of column and bearing corrosion are evaluated in this section. Consideration of the simultaneous effects of corrosion degradation of reinforcing bars in the RC columns and steel bridge bearing assembly reveals several interesting trends in the seismic response of the MSC steel girder bridge. Fig. 9 illustrates the seismic demands on the RC columns, expansion bearings in the longitudinal direction, and fixed bearings in the transverse direction due to joint consideration of the corrosion deterioration mechanisms of the columns, fixed and expansion bearing assembly. The 50- and 100-year corroded bridge responses are compared to the pristine, or time zero, bridge response using column moment-curvature and bearing force-displacement plots for the sample ground motion.

The corroded RC columns are found to show a consistent increase in the curvature ductility demand, which increases by 63 and 115% for the 50- and 100-year-old column, respectively, relative to the nondeteriorated column. It is interesting to note that as opposed to earlier findings when single component deterioration was considered, a joint consideration of column and bearing corrosion reveals a decrease in peak deformation demand on the fixed bearings in the longitudinal direction. Quantitatively, 11 and 44% reductions in peak deformations for the 50- and 100-year-old steel fixed bearings are observed along the longitudinal direction. This reduction in fixed bearing deformations is primarily attributed to the concentration of damage in the corroded columns and the dynamic response of the bridge deck and columns as a nearly single degree of freedom system, with little deformation occurring over the columns at the location of the fixed bearings. Additionally, there is a reduction in the expansion bearing deformations in the longitudinal direction due to continued increase in coefficient of friction due to debris accumulation along the service life of the bridge (Fig. 9). Consequently, reduced displacements in the longitudinal direction result in reduced pounding forces upon the closure of the 71-mm gap between the deck.
and the abutment. The decrease in pounding results in a respective 11 and 27% decreases in the passive deformation of the abutments for the 50- and 100-year-old corroded bridges.

Both fixed and expansion bearings are found to experience large demands in the transverse direction for both pristine and corroded bridge. Additionally, the increase in demand on the columns in the transverse direction is not as dramatic as that in the longitudinal direction and a reduced post yield stiffness of the corroded fixed and expansion bearings leads to higher peak bearing displacements as the bridge nears the end of its service life. For example, the fixed bearing deformations increase by approximately 15 and 110% [Fig. 9(c)], and the expansion bearings deformations increase by 13 and 69% in the transverse direction for the 50- and 100-year-old bridge. The impacts of these findings on the seismic fragility of the bridge system along its service life are discussed in the next section, considering uncertainty in the bridge and ground motion realizations.

Impact of Corrosion-Induced Component Deterioration on Seismic Fragility

The deterministic analysis for the corroded case study bridge served to help illustrate the bridge response characteristics under combined corrosion-induced deterioration and seismic loading. However, given the aleatoric and epistemic uncertainties inherent in the modeling parameters of the structure, corrosion process, and ground motion realizations, a full probabilistic analysis is required to ascertain the effect of corrosion on the probability of seismic bridge damage. Additionally, since the deterioration mechanisms are time-dependent processes and the corrosion parameters tend to increase in variability toward the end of the bridge’s service life, as previously shown in Figs. 2 and 4, the evolution of bridge fragility in time should be quantified in the probabilistic assessment.

Methodology for Development of Fragility Curves

Seismic fragility curves represent the probability of structural damage conditioned upon ground motion intensity and can provide insight on bridge seismic vulnerability at both the component and system levels. The generic expression of seismic fragility follows the form

\[ p_f = P(\text{demand} > \text{capacity}|\text{IM}) \]  

where IM=intensity measure of the ground motion, taken as PGA in this study, and the capacity varies for different damage states. Changes in structural performance due to corrosion inevitably result in variations in seismic vulnerability. Therefore, fragility curves are developed herein using a simulation-based analytical approach rooted in nonlinear time history analysis to capture the effect of aging on the seismic vulnerability of the MSC steel girder bridge. A total of 96 two component ground motions from the synthetic ground motion suites of Wen and Wu (2001) and Rix and Fernandez (2004) are used in the analysis, representative of a range of potential ground motions for the CSUS region. An equal number of three-dimensional bridge samples are generated through the Latin hypercube sampling, considering the potential uncertainty in structural, material, and corrosion related parameters at each point in time along the service life of the bridge. In addition to the corrosion related parameters presented above in Table 1, the probabilistic models for the random variables considered for the bridge structure include those previously identified for MSC steel bridges in the CSUS (Nielsen and DesRoches 2007a), namely, for concrete compressive strength, steel strength, stiffness of foundation piles, damping ratios, and gap between the deck and abutment. As previously noted, probability density functions (PDFs) for corrosion parameters that affect both the cross-sectional area of longitudinal reinforcement and column yield
curvature, and as well as the yield and ultimate strength of bearing components, are also integrated in the bridge sample generation.

Following a similar approach as presented in the component of Nielson and DesRoches (2007b) and Padgett and DesRoches (2008), system level fragility analysis is conducted. For the probabilistic analysis along the service life of the bridge, the 96 bridge samples generated for each point in time (e.g., 0, 25, 50, 75, and 100 years) are each subjected a seismic ground motion from the suite in a nonlinear time history analysis. Probabilistic seismic demand models are developed, which reflect the relationship between peak component demands and ground motion intensity. The responses considered in this research include such seismically vulnerable components as columns, fixed bearings, expansion bearings, and abutments. Consequently, the median value and logarithmic standard deviation of seismic demand at a particular point in time are calculated through regression analysis, where the median value is assumed to follow a power law model (Cornell et al. 2002)

\[
D(t)_{median} = r_1(t)IM^{r_2(t)}
\]

where \( r_1(t) \) and \( r_2(t) \) = regression parameters for the point in time under consideration (e.g., year) and IM = ground motion intensity. The correlation between component demands is also evaluated for use in the system fragility analysis.

In addition to the probabilistic model of seismic demand, the fragility analysis requires estimates of the structural capacities of the different bridge components. The limit state capacities used in this study are the lognormal capacity estimates presented by Nielson and DesRoches (2007a) for typical MSC steel girder bridges. The limit state capacities for each damage state (slight, moderate, extensive, and complete) are presented in Table 5 in the Appendix for reference. The use of curvature ductility demand for the column capacity limit state, as a maximum curvature normalized by the yield curvature, averts the need to explicitly change the capacity limit state models. However, the change in yield curvature and the reduced moment capacity for the corroded columns translate into a change in curvature ductility demands placed on the columns, as previously illustrated.

With the lognormal distributions for the structural demand and capacity, the lognormal distribution for the seismic fragility of the components can be found in closed form (Melchers 1999). Assessment of the bridge system reliability is carried out by assuming the bridge as a series system, wherein failure of a single component is representative of bridge failure, similar to the system representation adopted in structural reliability studies for bridges (e.g., Nielson and DesRoches 2007b; Nowak and Cho 2007) or other structural systems (Der Kiureghian and Dakessian 1998; Cimellaro et al. 2010). This system definition is consistent with the capacity estimates adopted in the paper, which offer the limit of component response upon which system level functionality inhibition is anticipated and implies that failure of any one of the components is indicative of overall bridge system failure. Under the series system assumption, the probability of the bridge system is at or beyond a particular failure limit state that is the union of the probabilities of each of the components being in the same limit state. This can be mathematically shown as

\[
P[\text{failure}_{\text{system}}] = \bigcup_{i=1}^{n} P[\text{failure}_{\text{component}}]
\]

where \( P[\text{failure}_{\text{system}}] = \text{probability of failure of the bridge system;}
\)

\( P[\text{failure}_{\text{component}}] = \text{probability of failure of the } i^{th} \text{ component;}
\)

and \( n = \text{total number of vulnerable bridge components. As previously noted, the estimate of correlation between peak component responses enables construction of a joint PDF for component demand. The bridge system fragility is then evaluated by comparing the joint PDF of demand with the component capacities for each damage state via the Monte Carlo analysis to derive system fragility estimates that account for component correlations at different points in time along the service life of the bridge. The overall corroded bridge system fragility can thus be mathematically represented as}

\[
P[DS|PGA] = \Phi \left( \frac{\ln(PGA) - \ln(m(t))}{\zeta(t)} \right)
\]

where \( m(t) \) and \( \zeta(t) \) = median values (in units of \( g \) PGA) and logarithmic standard deviations of the system fragilities at different points in time \( t \) along the service life and DS = damage state.

### Bridge Component and System Fragilities

Bridge component and system fragilities are evaluated at different points in time along the service life of the bridge to assess the effect of corrosion on the seismic vulnerability. It is observed that at the component level, while there is a steady increase in the fragility of certain elements, some other components show a reduced vulnerability with time. This contradicting trend in the component seismic fragilities is consistent with the findings from the deterministic analysis which revealed that increased demands on typical components (such as deteriorated RC columns) result in decreased demands on certain other bridge elements (such as fixed bearings in the longitudinal direction). Due to space limitations, changes in fragilities of other bridge components, whether increasing or decreasing, for the aged bridge relative to...
the pristine bridge are presented qualitatively in Table 2. For the RC columns, fixed and expansion bearings, these trends are found to be consistent for all damage states at different points in time along the service life of the bridge. The percent change in the median value PGA for the complete damage state after 75 years is also listed for each component. This quantity provides insight on the relative magnitude of change in fragility due to corrosion for each bridge component. The abutment fragilities of the pristine along both directions are insignificant beyond the moderate damage state and correspondingly the changes in fragility in the corroded bridge are found to be negligible.

Although consideration of joint degradation of RC columns and steel bridge bearings results in increasing fragility of some components (for example, columns) and decreasing fragility of others (for example, fixed bearings), the overall seismic fragility at the system level increases in time as the bridge continues to corrode (Fig. 10). For example, for a ground motion having PGA=0.6 g, there is a 30% chance of achieving complete damage for the pristine, or time zero, bridge, but after 75 years of exposure to deicing salts, the chance of complete damage for the same level of earthquake is 49%. A complete list of the median and dispersion values of the system fragility at all damage states is provided in Table 3 for different points in time. The decrease in median values of fragility for the different damage states along the service life of the deteriorated bridge is further a direct indication of the increased bridge system vulnerability due to corrosion of its critical structural components. In general, there is also a slight change in the dispersion over time, indicating reduced uncertainty in estimating the PGA value corresponding to exceedance of each damage state when the bridge is corroded. The overall increase in seismic fragility of the bridge can be attributed to the dominance of the columns, transverse and longitudinal expansion bearings, followed by the transverse fixed bearings, indicating the bridge system vulnerability. On the whole, the seismic vulnerability of these components tends to be negatively affected by the continued corrosion of the bridge.

Table 3. Median and Dispersion Values of System Fragilities for All Damage States at Different Points in Time

<table>
<thead>
<tr>
<th>Time (years)</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 years</td>
<td>m=0.269</td>
<td>m=0.517</td>
<td>m=0.657</td>
<td>m=0.888</td>
</tr>
<tr>
<td></td>
<td>ζ=0.701</td>
<td>ζ=0.621</td>
<td>ζ=0.648</td>
<td>ζ=0.7</td>
</tr>
<tr>
<td>25 years</td>
<td>m=0.266</td>
<td>m=0.48</td>
<td>m=0.608</td>
<td>m=0.789</td>
</tr>
<tr>
<td></td>
<td>ζ=0.646</td>
<td>ζ=0.582</td>
<td>ζ=0.621</td>
<td>ζ=0.64</td>
</tr>
<tr>
<td>50 years</td>
<td>m=0.261</td>
<td>m=0.467</td>
<td>m=0.596</td>
<td>m=0.788</td>
</tr>
<tr>
<td></td>
<td>ζ=0.607</td>
<td>ζ=0.544</td>
<td>ζ=0.586</td>
<td>ζ=0.617</td>
</tr>
<tr>
<td>75 years</td>
<td>m=0.235</td>
<td>m=0.395</td>
<td>m=0.508</td>
<td>m=0.674</td>
</tr>
<tr>
<td></td>
<td>ζ=0.542</td>
<td>ζ=0.53</td>
<td>ζ=0.563</td>
<td>ζ=0.596</td>
</tr>
<tr>
<td>100 years</td>
<td>m=0.208</td>
<td>m=0.35</td>
<td>m=0.455</td>
<td>m=0.634</td>
</tr>
<tr>
<td></td>
<td>ζ=0.526</td>
<td>ζ=0.492</td>
<td>ζ=0.537</td>
<td>ζ=0.582</td>
</tr>
</tbody>
</table>

Note: m=median (PGA) and ζ=dispersion.
for the fragility parameters for all of the damage states are given in Table 4. These coefficients provide time-dependent models for the median and dispersion that can be substituted in the following equation to evaluate the bridge system fragility at any point in time considering column and bearing corrosion

\[ P[DS|PGA](t) = \Phi \left( \ln(PGA) - \ln(p_{1,m}t^2 + p_{2,m}t + p_{3,m}) \right) / \left( p_{1,\zeta}t^2 + p_{2,\zeta}t + p_{3,\zeta} \right) \] (18)

where the subscripts on the coefficients indicate median, \( m \), or dispersion, \( \zeta \).

The format of time-dependent functions presented in this paper offers an efficient approach to capture the impact of aging and deterioration on bridge system fragility, thus aiding bridge owners and managers in assessing evolving seismic risk at various stages of bridge service life. Furthermore, by identifying a tractable time-varying quadratic format for fragility parameters this method can be easily integrated into regional risk assessment and loss estimation software. The fragility parameters and corresponding fragility curves derived in this paper reflect the time-dependent seismic fragility of the representative MSC steel bridge with corrosion deterioration models as discussed in the previous sections and are not intended for application to portfolios of structures. However, the approach presented herein illustrates the importance of integrating the deterioration of key structural components, such as columns and bearings, influence on dynamic behavior, and propagation to affect bridge component and system fragility. Furthermore, the methodology for analysis and time-dependent seismic fragility format, including assessment of quadratic models for time-variant lognormal fragility parameters, can be applied in the future to additional bridge types and classes, or portfolios, of bridges with gross geometric variation.

Additionally, as outlined by Gardoni et al. (2002), the ability to incorporate experimental or field data as it emerges can offer a valuable opportunity to improve the accuracy of bridge vulnerability estimates, particularly for models targeted at bridge-specific fragility analysis. Over the years, the Bayesian framework has emerged as a potential tool for fragility estimation of structures (Singhal and Kiremidjian 1998; Der Kiureghian 2002; Gardoni et al. 2002; Berahman and Behnamfar 2007; Graf et al. 2009). Future opportunities exist for exploiting such approaches as the Bayesian techniques for updating the time-dependent component and system fragility models with field data, such as improved estimates of corrosion modeling parameters. Similarly, multidimensional fragility curves or fragility surfaces (Gardoni et al. 2002; Koutsourelakis 2010) can also be developed as a part of future research endeavors exploiting the time-dependent fragility format and propagation of joint component aging phenomena presented in this study.

### Table 4. Coefficients of Quadratic Interpolation for the Median and Dispersion Values at Different Damage States

<table>
<thead>
<tr>
<th>Damage state</th>
<th>Median</th>
<th>Dispersion</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( p_{1,m} )</td>
<td>( p_{2,m} )</td>
</tr>
<tr>
<td>Slight</td>
<td>(-9.8 \times 10^{-6})</td>
<td>(-0.0002)</td>
</tr>
<tr>
<td>Moderate</td>
<td>(-1.5 \times 10^{-5})</td>
<td>(-0.0006)</td>
</tr>
<tr>
<td>Extensive</td>
<td>(-1.8 \times 10^{-5})</td>
<td>(-0.0009)</td>
</tr>
<tr>
<td>Complete</td>
<td>(-9.7 \times 10^{-6})</td>
<td>(-0.0023)</td>
</tr>
</tbody>
</table>
Conclusions

This paper provides a framework for time-dependent fragility analysis of corroded bridge performance in seismic events, illustrating the impact of aging on component and system reliability. A case study MSC steel girder bridge representative of median geometric characteristics of the CSUS bridge class is considered in the seismic performance assessment though variability in structural component, material, and corrosion modeling parameters that are explicitly modeled. Susceptibility to corrosion attacks from the application of chloride-laden deicing salts commonly used across the United States is considered in the simulation of bridge aging and deterioration. The degradation mechanisms considered include the corrosion deterioration of RC columns due to reinforcement area loss, the deterioration of steel bridge bearing assemblies due to corrosion of steel anchor bolts, the transverse keeper plates, and the buildup of corrosion debris. The deterioration of these components affects the lateral force resisting system of bridges under seismic loading, resulting in reduced moment capacity and yield curvature of the columns, reduced ultimate lateral strength of the fixed and expansion bearings, and increased coefficient of friction in the bearings due to debris accumulation. The nonlinear time history analysis illustrates that when the deterioration of bridge components is considered individually, there is a significant shift in the dynamic response of the bridge producing an increase in the seismic demand on the individual components. However, joint consideration of the component corrosion effects reveals that while the seismic demand on some components (e.g., RC columns and expansion bearings) shows a steady increase along the service life of the bridge, there is a decrease in the demand on some components such as the fixed bearings in the longitudinal direction.

A full probabilistic analysis is conducted to evaluate the seismic fragility of the MSC steel bridge given uncertainty in bridge, ground motion, and corrosion parameters. The bridge system fragility curves for each damage state reveal a significant increase in the bridge system vulnerability over time due to aging. For example, after 75 years of exposure to chlorides, the median value PGA for the complete damage state decreases by 27%. Simple quadratic functions are introduced to enable easy assessment of the time-dependent shift in median and dispersion values of seismic fragility for each damage state due to corrosion. The findings highlight the importance of considering the effects of aging and deterioration on the seismic vulnerability of bridges. Moreover, the time-dependent fragility format presented provides a straightforward approach to incorporate the influence of bridge age and exposure condition within seismic risk assessment packages. Such models offer more realistic estimates of corroded bridge seismic vulnerability and enable more accurate estimates of potential damage, life cycle cost, and needed rehabilitation. The paper has also highlighted opportunities for future work, including field-condition informed updating of the fragility curves or multidimensional fragility surfaces conditioned upon various structural or aging parameters to improve the analysis for bridge-specific fragility parameters. Given the significance of this study’s findings, future work should also address time-dependent fragility of additional bridge types and geometries, generalized classes of bridges, as well as deterioration mechanisms.

Acknowledgments

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Appendix

The capacity limit states adopted for each bridge component are listed in Table 5.

References


Table 5. Capacity Limit States for Different Bridge Components for the Chosen Bridge (Adapted from Nielson and DesRoches 2007a)

<table>
<thead>
<tr>
<th>Component</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median</td>
<td>Dispersion</td>
<td>Median</td>
<td>Dispersion</td>
</tr>
<tr>
<td>Columns</td>
<td>1.29</td>
<td>0.59</td>
<td>2.10</td>
<td>0.51</td>
</tr>
<tr>
<td>Fixed bearings—longitudinal</td>
<td>6.0</td>
<td>0.25</td>
<td>20.0</td>
<td>0.25</td>
</tr>
<tr>
<td>Fixed bearings—transverse</td>
<td>6.0</td>
<td>0.25</td>
<td>20.0</td>
<td>0.25</td>
</tr>
<tr>
<td>Expansion bearings—longitudinal</td>
<td>37.4</td>
<td>0.60</td>
<td>104.2</td>
<td>0.55</td>
</tr>
<tr>
<td>Expansion bearings—transverse</td>
<td>6.0</td>
<td>0.25</td>
<td>20.0</td>
<td>0.25</td>
</tr>
<tr>
<td>Abutment—passive</td>
<td>37.0</td>
<td>0.46</td>
<td>146.0</td>
<td>0.46</td>
</tr>
<tr>
<td>Abutment—active</td>
<td>9.8</td>
<td>0.70</td>
<td>37.9</td>
<td>0.90</td>
</tr>
<tr>
<td>Abutment—transverse</td>
<td>9.8</td>
<td>0.70</td>
<td>37.9</td>
<td>0.90</td>
</tr>
</tbody>
</table>


