Evaluation of Seismic Demand on Bridge Nonstructural Components Using ASCE 7

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Abstract. Bridge nonstructural components, also known as bridge appurtenances or attachments, are not part of the load resisting systems of a bridge structure. Examples of bridge appurtenances include parapets, emergency walkways, Bridge Utility Systems (BUS), signs, and lighting posts attached to the bridge deck. Traditionally, these attachments are designed to resist wind load, live load, and vehicle impact load. However, while these loading types may be thought to control the design, damage to bridge appurtenances in past large earthquakes, such as failure of utility poles and signs, and falling of mounted masts, shows that more attention from the design and research community may be warranted. Current bridge design codes and state Department of Transportation (DOT) provisions do not address the seismic design of nonstructural components. Additionally, the existing AASHTO LRFD specifications for structural supports for signs, luminaires, and traffic signals focus primarily on wind design and fatigue performance and does not include provision for seismic loads. The designer instead is referred to project specific guidelines which typically do not exist. The objective of this paper is to raise awareness on the current state of the practice and discuss possibilities toward a unified approach for seismic design and performance assessment of bridge nonstructural components. As there are no clear procedures for evaluating bridge structural components for seismic loads, it is acceptable to seek guidelines in building codes such as ASCE 7. This paper compares two editions of ASCE 7 and presents two case studies to demonstrate their applicability to the seismic evaluation of bridge nonstructural components. The paper concludes with recommendations and suggestions for future research.

Keywords: Bridge appurtenances, seismic design, code review, seismic demand, earthquake damage.

1. INTRODUCTION

Bridge nonstructural components, also known as bridge appurtenances or attachments, although not part of the load resisting system, have an important role in maintaining bridge functionality. Examples of nonstructural components on bridges include light poles/luminaries, transmission lines, emergency walkways, and Bridge Utility Systems (BUS). Typically, these components are designed to resist wind load, live load, and vehicle impact load. Seismic loads are often not considered as the governing demand, and therefore, may not be explicitly considered. However, damage to bridge nonstructural components in past large earthquakes suggest that these components could be vulnerable to seismic loads and that their performance, as well as impact on the bridge behaviour, need to be investigated. Even if the bridge load-carrying capacity is not compromised, damage such as failure of utility poles and signs, or falling of mounted masts, could delay the functional recovery of the bridge, not only resulting in economic losses but potentially limiting access to food, supplies, and medical attention. In addition, rescue operations could be compromised due to the inability to use the bridge. The bridge and many of the systems it carries are lifelines for the local community and are worth consideration when assessing local seismic resilience.

The objective of this paper is to review the current state of the practice and to discuss a possible unified prescriptive approach for seismic design and evaluation of bridge nonstructural components based around the ASCE 7 framework. We present an extensive literature review and compile a summary of seismic design guidelines from relevant codes, standards, and design criteria, including AASHTO and amendments by different state's Department of Transportation (DOT), and building codes such as ASCE 7. Two case studies are presented to investigate the use of ASCE 7 in seismic demand evaluation of representative bridge nonstructural components, with a focus on comparing the ASCE 7-16 approach to the updated method in ASCE 7-22 and to numerical analysis results. The paper concludes with recommendations and suggestions for future research.

2. PAST SEISMIC PERFORMANCE OF BRIDGE APPURTENANCES

There have been numerous reports of damage to bridge nonstructural components following seismic events. According to Siringoringo *et al* [2020], more than one thousand lighting and utility poles around the Hanshin Expressway were damaged during the 1995 Great Hanshin-Awaji (Kobe) Earthquake (M6.9). Images from the event show yielded barrier mounted light posts and toppled poles over elevated highways. Abé and Shimamura [2014] reported that over 504 electrical power poles and over 10 transformers were damaged following the 2011 Tōhoku Earthquake (M9.0). Some of the damaged poles were mounted on the Shinkansen railway bridge. Seismic damages of BUS, such as potable and wastewater utility lines typically mounted on bridge decks, have also been reported. During the 2010-2011 Canterbury Earthquake Sequence (CES) in New Zealand, severe damage to utility lines was reported despite good structural performance of the bridge structures [Rais *et al* 2015; Palermo *et al,* 2011]. Although the principal source of damage during CES was identified as rotation of the abutments at deck-abutment interface, other damage mechanisms to bridge-mounted utility lines have been recognized. Examples include failure of the pipeline at midspan during the CES and buckling of pipeline during 1994 Northridge Earthquake [Rais *et al*, 2015; Schiff, 1997]. Images from EERI's Virtual Clearinghouse [2016] documenting the Kaikoura Earthquake (M7.5) also show damages to BUS.

The survey of damages discussed above suggests that nonstructural components on bridges are susceptible to earthquakes, and thus, warrant more attention from the engineering community. While seismic behaviour of *building* nonstructural components and their importance to seismic resilience is relatively well-understood, there seems to be a lack of a consensus on seismic design methodologies of *bridge* nonstructural attachments. In addition, while seismic design of bridge *structural* components is relatively well-researched and documented, such as in AASHTO code and other standards by State DOTs, it is the authors' opinion that similar consideration for bridge *nonstructural* components has not been adequately given.

3. STATE OF THE PRACTICE IN SEISMIC DESIGN OF BRIDGE NONSTRUCTURAL COMPONENTS

Current bridge design code [AASHTO, 2020] and/or state DOTs provisions do not specifically address the seismic design of nonstructural components. Additionally, the existing LRFD specifications for structural supports for signs, luminaires, and traffic signals [AASHTO, 2015] focus primarily on wind design and fatigue performance and does not include provisions for seismic loads. Instead, the designer is referred to project-specific seismic guidelines, which may not exist, or which refer to codes or standards intended primarily for building structures. Agencies such as the California Department of Transportation (Caltrans), although having strict requirements and extensive guidelines for seismic design and performance of bridge structural components, does not provide any recommendations for the seismic design of bridge nonstructural components. Similarly, the academic community offers limited research on bridge nonstructural components under seismic loads. Siringoringo *et al* [2020] performed a numerical study on the seismic behaviour of a tapered light pole mounted on a highway bridge. Results show that if the fundamental frequency of the bridge is within the range of ±30% of the fundamental frequency of the light pole, resonance is observed, resulting in larger seismic demand and potential bending failure. Bharil *et al* [2001] proposed general guidelines for bridge water pipe installation including design loads and safety factors for pipe hangers. The paper conservatively recommended using 0.5g for the acceleration coefficient in any lateral direction for the seismic design of these components; however, complete seismic design guidelines were not developed.

In the absence of clearer bridge-specific guidance for seismic design of bridge appurtenances, it is the authors' experience that seismic design is commonly: 1) ignored, under the perhaps incorrect assumption that other design loads and detailing requirements will govern, 2) based on seismic loads used for the base structure, which may ignore possible dynamic interaction between the components and the base structure and the design ground motions, 3) estimated analytically, which can be inaccurate and/or time consuming depending on the method used, or 4) based upon prescriptive provisions in the building code, which may require some judgment to apply to bridges. While numerical analysis may always be the most accurate approach, given that many smaller bridges continue to be designed for earthquakes using prescriptive methods, establishing consensus around a reliable prescriptive method for practitioners would be valuable. We note that Goel [2018] has identified a similar lack of prescriptive guidance for piers and wharves.

As there are no clear bridge-specific prescriptive guidelines for evaluating nonstructural components for seismic loads, it is common to seek guidance in building codes, such as ASCE 7, which contain more robust guidelines for seismic design of nonstructural components. These provisions are frequently referenced (often via reference to local building codes that reference them) by owners in project-specific design criteria and may be applicable (or require consideration alongside AASHTO or other provisions) to design of many nonstructural aspects of vehicular and pedestrian bridges. While ASCE 7 clarifies that the provisions are applicable to building and non-building structures, the provisions and equations were developed in the context of buildings, and therefore their application to bridges and other nonbuilding structures may require some judgment by the designer. However, it is the authors' opinion that ASCE 7 currently provides the best available *framework* for prescriptive seismic design of nonstructural components in general, and the remainder of this paper is dedicated to investigating the potential application of these provisions in the context of bridge design, including key considerations, assumptions, and limitations that may affect the resulting designs.

4. A REVIEW OF ASCE 7 PROVISIONS

The general approach in ASCE 7 is to determine the effective horizontal seismic design force, F_p, acting on the nonstructural component as a function of design peak ground acceleration (PGA). The provisions had been largely unchanged from the time they first appeared in their modern form in ASCE 7-98, through to the version referenced by the building code at time of writing, ASCE 7-16. Using ASCE 7-16, F_p is defined as:

$$F_p = 0.4S_{DS}\left(1 + 2\frac{z}{h}\right)\frac{a_p}{\left(\frac{R_p}{I_p}\right)}W_p \qquad \text{[ASCE 7-16, Ch. 13] (1)}$$

where, S_{DS} = short period spectral acceleration, a_p = component amplification factor, I_p = component importance factor, R_p = component response modification factor, z = structure height at the component attachment level, h = average roof height, and W_p = component operating weight.

Continued poor performance of some nonstructural components and increasing desire for expedited functional recovery has led to a desire for further study and refinement of ASCE 7 provisions [ATC, 2017]. The National Institute for Science and Technology (NIST) GCR 13-917-23 report [Hooper *et al*, 2013] recognized nonstructural components as a crucial area of improvement. This led to further studies including NIST GCR 17-917-44 report [ATC, 2017] which provided recommendations for future work on seismic analysis and design of nonstructural components, as wells as NIST GCR 18-917-43 report [ATC, 2018] which resulted in revisions to existing seismic provisions for nonstructural components in building structures (i.e., ASCE 7-22).

The primary change between the 7-16 and 7-22 provisions is that the equations for deriving F_p were modified to explicitly account for the dynamic characteristics of the structure and its interaction with the nonstructural component. Using ASCE 7-22, F_p is defined as:

$$F_p = 0.4S_{DS}\frac{H_f}{R_\mu}\frac{C_{AR}}{R_{po}}I_pW_p \qquad \text{[ASCE 7-22, Ch. 13] (2)}$$

where, H_f = factor for force amplification as a function of structure height, C_{AR} = component resonance ductility factor, $R_\mu = [1.1R/I_e\Omega_o]^{1/2}$ is the structure ductility factor which shall be greater than 1.3, R_{po} = component strength factor. The factors in both equations are either tabulated in the code or are determined based on the structure and component dynamic properties. The factor H_f is evaluated as follows:

$$H_f = 1 + a_1\left(\frac{z}{h}\right) + a_2\left(\frac{z}{h}\right)^{10} \; where \; a_1 = \frac{1}{T_a} \leq 2.5 \; and \; a_2 = \left[1 - \left(\frac{0.4}{T_a}\right)^2\right] \geq 0 \qquad \text{[7-22] (3)}$$

$$H_f = 1 + 2.5\left(\frac{z}{h}\right) \; when \; structure \; period \; is \; unknown \qquad \text{[7-22] (4)}$$

where T_a is the period of the structure[a]. The code provides Equation (4) as an alternative way to determine H_f if the period of the structure is not readily available. In calculating R_μ, the following factors are defined for the structure: R = response modification factor, I_e = importance factor, and Ω_o = overstrength factor.

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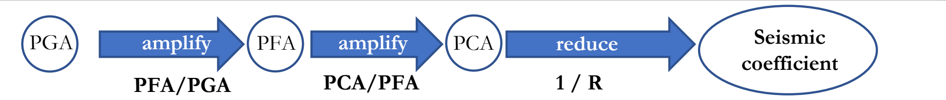
[a] ASCE 7 defines T_a as the approximate period as determined using the empirical equations of ASCE 7 (which often yield higher periods than would be determined by analysis), but for nonbuilding structures ASCE 7 does allow T_a to be determined based on a "properly substantiated analysis."

Assuming that it is unreasonable to design for a force that is excessively low or high, both Equations (1) and (2) are limited to minimum and maximum values of $0.3S_{DS}I_pW_p$ and $1.6S_{DS}I_pW_p$, respectively.

Figure 1 breaks down Equations (1) and (2) above and compares the "equivalent" terms. The first term is the PGA for the design earthquake, estimated as $0.4S_{DS}$ in both versions of ASCE 7, which accounts for the intensity of ground shaking. The second term is the amplification factor from PGA to peak floor acceleration (PFA), which accounts for the dynamic properties of the building. The third term is the component amplification factor from PFA to peak component acceleration (PCA), which accounts for the dynamic properties of the component [ATC, 2018]. These amplification factors convert the ground acceleration to that acting on the component. The last term is associated with the reduction factor, R, which is used to account for energy dissipation due to component and structure nonlinear behaviour. As illustrated in Figure 1, the total seismic coefficient, F_p/W_p, for any nonstructural component is defined as PGA x (PFA/PGA) x (PCA/PFA) x (1/R).



$0.4S_{DS}$ ×	$\left(1+2\frac{z}{h}\right)$	×	a_p	×	$\frac{1}{R_p}$	=	$\frac{F_p}{W_p}$	**ASCE 7 - 16**
$0.4S_{DS}$ ×	H_f	×	C_{AR}	×	$\frac{1}{R_\mu}\frac{1}{R_{po}}$	=	$\frac{F_p}{W_p}$	**ASCE 7 - 22**

Figure 1. Conceptual framework for ASCE 7 nonstructural component demand

We acknowledge that this may not reflect how the provisions were actually developed and the terms may not separate as cleanly in practice. For example, in theory, a_p and C_{AR} could approach infinity for structures where the component approaches resonance with the supporting structure and ground motion. However, ASCE 7 commentary [ATC, 2018] implies that a_p and C_{AR} are not purely related to structural dynamics, and both consider component damping and ductility, despite the fact that there is a separate component force reduction parameter. Thus, while separating the equation into the terms described here provides a logical framework for comparing each version of ASCE 7 to our numerical results, both the individual terms and final result of each methodology should be compared. We also note that while the ASCE 7 provisions have a basis in structural dynamic theory, certain assumptions need to be made to create simple equations that produce reasonably economical designs with reliable performance. According to NIST GCR 18-917-43 report [ATC, 2018], instrumentation and analytical data from building structures was used to develop some of the coefficients and set certain bounding values. These underlying assumptions and their possible impact on the results (i.e., conservative versus unconservative) needs to be considered when comparing prescriptive results to pure analysis, and when considering the application of this methodology to non-building structures or even for buildings with relatively unusual characteristics. The relevant assumptions and possible impacts to results will be discussed in this paper.

The following sections will present examples of how each of these terms might be defined by the practicing bridge designer, either prescriptively or numerically, for an archetype bridge and compare the component seismic demands based on guidelines from ASCE 7-16 and ASCE 7-22, and from a linear time history analysis.

5. CASE STUDIES

This section investigates application of ASCE 7-16 and 7-22 provisions through two case studies: a bridge-mounted pipeline and a bridge-mounted light pole. The components are assumed to be mounted on a selected archetype bridge, which is discussed in the following sub-section.

5.1 ARCHETYPE BRIDGE STRUCTURE

Caltrans Seismic Design Criteria (SDC) [Caltrans, 2019] defines three categories of bridges based on the expected post-earthquake damage state and service level (namely ordinary, recovery, and important bridges). The archetype bridge structure in this study, shown in Figure 2, is an ordinary cast-in-place concrete box girder bridge from Caltrans Bridge Design Practice (BDP) manual [Caltrans, 2015]. The bridge structure has three continuous spans with lengths of 126 ft, 168 ft, and 118 ft, respectively. The superstructure is composed of a 6.75 ft deep multi-cell box girder and the substructure includes two bents, each with two 6-ft diameter columns that are 44 ft tall.

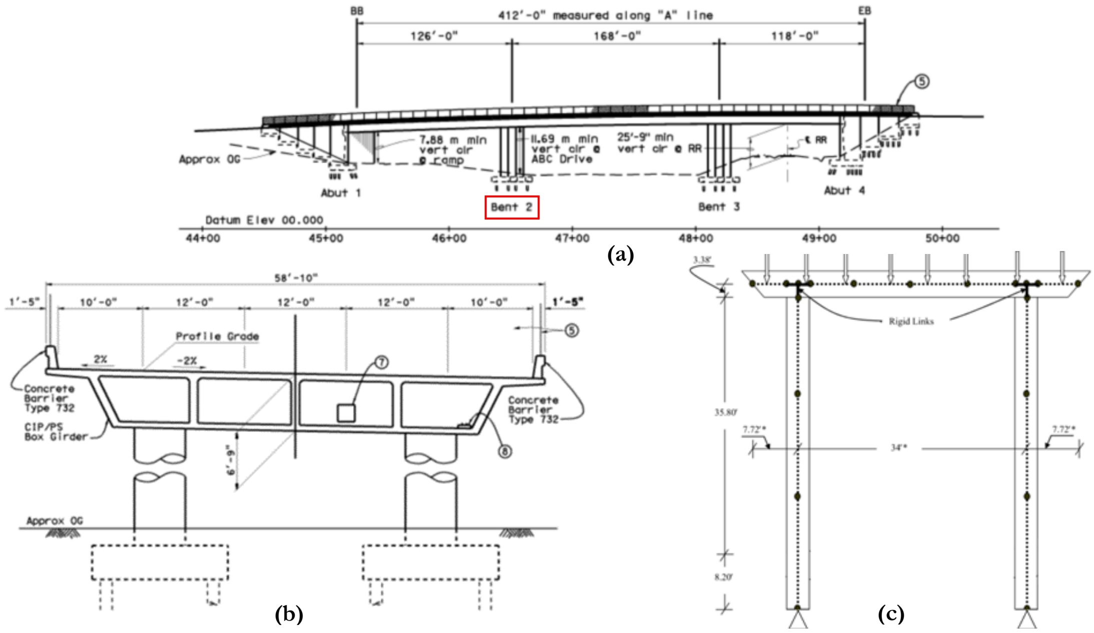


Figure 2. Archetype bridge: (a) elevation view, (b) section view, and (c) structural analysis model [Caltrans, 2015]

5.2 ARCHETYPE NONSTRUCTURAL COMPONENTS

The study considered two archetype nonstructural components: a bridge-mounted pipeline and bridge-mounted light pole. Bridge-mounted utility pipelines can be supported from the bridge deck in various ways. The archetype considered in this paper is a 147 ft long AWWA C151 ductile iron pipeline with a nominal diameter of 20 inches that was installed on Bethel Island Bridge in Northern California [Brick and Tilden, 2019]. The pipeline is supported vertically at every 10 ft by a trapeze that consists of two vertical all thread hanger rods and one horizontal HSS 3x3x1/4. One of the vertical rods is stiffened using Unistrut P1000, and a Unistrut P1000 kicker for lateral bracing is provided every 20 ft. The archetype light pole considered in this paper is the Type 21 pole per Caltrans Standard Plans [Caltrans, 2018]. This pole has a height of 35 ft with a projected arm catching the lighting fixture at 8 ft from centre of gravity. The cross-section is a circular tube tapered linearly along the height with a base diameter of 8.6 inches and top diameter of 3.6 inches. The pole supports 21 lb luminaire per Caltrans authorized materials list.

As summarized in Figure 1, the seismic design force F_p depends on various coefficients typically given in the code. Table 1 summarizes these factors for the two archetype components as given in both ASCE 7-16 and 7-22. Note that the code does not provide factors for poles as they are not typically found in buildings. Thus, the values in Table 1 correspond to an alternative component with similar dynamic properties.

Table 1. ASCE 7 Seismic coefficients for pipe and pole components

Component	Equivalent ASCE 7 Component	Edition	a_p or C_{AR}	R_p or R_{po}	a_p/R_p or C_{AR}/R_{po}
Pipe	Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	ASCE 7 – 16	2.5	9	0.28
		ASCE 7 – 22	1	2	0.50
Pole	Other flexible architectural components – High deformability elements and attachments	ASCE 7 – 16	2.5	3.5	0.71
		ASCE 7 – 22	1.4	1.5	0.93

5.3 ANALYSIS

In this paper the seismic demand on the archetype nonstructural components is estimated using the following two approaches: 1) prescriptive code-based method using both ASCE 7-16 and ASCE 7-22, and 2) a linear time history analysis. First, the similar terms in the two prescriptive code-based equations are compared. Then, the ASCE 7 results are compared with the more detailed numerical analysis.

An important part of the seismic evaluation of both structural and nonstructural components is the fundamental period. The periods for the archetype bridge, pipeline, and pole are estimated through modal analysis of linear elastic models performed in SAP2000. Sensitivity analysis indicated that the component's periods varied relatively little under reasonable design configurations. Thus, their periods were held constant for this study. However, bridge stiffness can vary significantly based on structural system, anticipated ductility, span length, superstructure/substructure connectivity, etc. Hence, to better understand how the seismic demand changes based on the stiffness of the structure, several fundamental periods of the bridge were investigated through modification of the bridge lateral stiffness, both higher and lower than the baseline period of 2.12 (Case 2) estimated from the Caltrans BDP example discussed above. Table 2 summarizes the component and bridge periods considered in this case study.

Table 2. Fundamental periods of bridges and components considered

	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Fundamental period of bridge, T_s (s)	2.98	2.12	1.49	1.22	0.94	0.67
Fundamental period of pipe component, T_p (s)			0.06 (All Cases)			
Fundamental period of pole component, Tp (s)			0.96 (All Cases)			

To further compare the results from the prescriptive equations per ASCE 7, a dynamic study of the bridge-component systems was performed. The natural periods of the archetype components were used to create equivalent lumped mass models that were mounted on the archetype bridge. Ground motions were selected and scaled to Caltrans 2014 Safety Evaluation Earthquake design spectrum for an arbitrary site located in downtown San Francisco (V_{s30} = 270 m/s, Site Class D). A linear time history analysis (THA) using a single ground motion tightly scaled to the design spectrum was performed for each bridge-component system to estimate the resulting maximum accelerations at top of deck and component centre of gravity, which correspond to PFA and PCA, respectively. This permitted the calculation of PFA/PGA and PCA/PFA ratios that are compared to ASCE 7 tabulated values.

We note that ASCE 7-22 Section 13.3.1.5 allows the use of nonlinear time history analysis in lieu of the prescriptive approach. The code requires that the mean demands from a suite of at least seven ground motions be used rather than the single time history used here. Where the component is not explicitly modelled, the PFA is multiplied by C_{AR}/R_{po} to determine the component demands. Because we included the component but modelled the component and structure as elastic, we would consider dividing PFA by the product of R_{po} and R_{μ}.

5.4 ANALYSIS RESULTS AND DISCUSSION

5.4.1 ASCE 7-16 vs. ASCE 7-22

5.4.1.1 *PFA/PGA Amplification Factor*

There is considerable difference in the PFA/PGA amplification factor between ASCE 7-16 and ASCE 7-22. ASCE 7-16 implies that PFA increases linearly based on the point of attachment of the component compared to the height of the structure, up to a maximum of 3 times PGA at the top of structure. In contrast, ASCE 7-22 increases PFA in a more nuanced manner depending on the dynamic characteristics of the structure. PFA depends on the type of lateral resisting system in addition to the mass or stiffness distribution along the height of the structure. ASCE 7-22 attempts to capture these key features by incorporating the period of the structure into the equation. In both ASCE 7 versions, PFA/PGA includes the ratio z/h. Since bridge components are typically mounted at deck level, which could be considered equivalent to "roof level" in a building, it might be natural for the designer to set z/h to unity. Since ASCE 7-16 does not explicitly account for the period of the structure, PFA/PGA has a constant value of 3 when z/h = 1. For ASCE 7-22, PFA/PGA varies and Table 3 summarizes this factor for the range of structure fundamental periods.

Table 3. PFA/PGA per ASCE 7-22

	Fundamental period of bridge, T_s (sec)					
	2.98	**2.12**	**1.49**	**1.22**	**0.94**	**0.67**
PFA/PGA at z/h = 1 (Equation 3)	2.3	2.4	2.6	2.7	2.88	3.1

Note that these are generally less than the fixed value of 3 prescribed by ASCE 7-16 for the period range studied but begin to creep higher than 7-16 for short-period bridges. Equation (3) implies that as the period of the structure approaches 0.4 sec, ASCE 7-22 approaches a maximum amplification of 3.5. Note that an alternate approach to estimate PFA/PGA per ASCE 7-22 is Equation (4), which yields a constant amplification factor of 3.5.

The basis for the PFA/PGA factor in both ASCE 7-16 and ASCE 7-22 is regression on data from instrumented buildings of various heights and structural systems, which ultimately reflects the dynamic characteristics of the structure, such as period, mode shape, and higher mode effect (and likely inherent damping and energy dissipation) [ATC, 2018]. Thus, if the dynamic properties of a given bridge approximate those of buildings (e.g., long- or multiple-span bridges or those whose behaviour is controlled by deck flexibility), ASCE 7 may be valid for estimating amplification, with ASCE 7-22 offering a more nuanced approach. However, for dynamically simple bridges (e.g., rigid deck bridges controlled by bent/pier flexibility) that behave more like a single degree of freedom (SDOF) structure, this amplification may be highly conservative. Therefore, the definition of PFA/PGA, including the assumption that z/h equals unity when the component is mounted on bridge decks, warrants further investigation.

5.4.1.2 *PCA/PFA Amplification Factor*

In ASCE 7-16, the PCA/PFA factors are a step function of either: 1.0 when the component is considered "rigid" (i.e., period less than 0.06s) or 2.5 when the component is considered "flexible" (i.e., period above 0.06s). The updated expression for PCA/PFA factor in ASCE 7-22 still accounts for this ratio of component and building period but in a more nuanced way. In addition to the archetype C_{AR} values provided in Table 1, ASCE 7-22 provides C_{AR} values of 1.0, 1.4, 1.8, 2.2, or 2.8, depending on the ductility and likelihood of the component being in resonance with the supporting structure [ASCE, 2022]. For the archetype bridge structure and components, from Table 1, it is evident that ASCE 7-22 component amplification factors are lower than ASCE 7-16 for both cases considered. We note, however, that values of R_{po} from ASCE 7-22 are also generally lower compared to corresponding values of R_p from 7-16, as discussed further in the next section.

According to ASCE 7, the formulation of a_p and C_{AR} factors accounts for component damping and ductility and is intended to be independent of the supporting structure properties. However, the factors are also intended to limit the design amplification by reducing the probability of component resonance to an acceptably low value [ATC, 2018], and thus have some inherent dependence on the ratio of component period to structure period. Like the PFA/PGA amplification factor, statistical analysis on instrumental data of buildings of various heights and structural systems formed part of the basis for these coefficients (along with properties of representative nonstructural elements). While nonstructural components on both bridges and buildings may have similar properties, the two types of supporting structures may vary in terms of what is considered "typical" and the dynamic properties of each. Survey of documents suggests that typical bridge lateral periods may range from 0.1 sec to 1.2 sec [Dusseau and Dubaisi, 1993; Zelaschi *et al*, 2016; Kuribayashi and Iwasaki, 1973; Feng *et al*, 2011], which is consistent with low- to mid-rise buildings and close to the mean period of components considered in ASCE 7 formulation (which have values of 0.33 sec for flexible and 0.12 sec for rigid components) [ATC, 2018]. Longer span bridges and tall buildings would both likely have periods much longer than this range but would both represent a relatively minor portion of the statistical population. Therefore, it would seem that the building data used to develop the ASCE 7 component amplification factors might still be a good fit for bridges when assessing probability of resonance, but this would warrant further research.

5.4.1.3 *Strength and Ductility Reduction Factor*

ASCE 7-16 provides a single response modification factor, R_p, which accounts for both ductility and overstrength in the component. In contrast, ASCE 7-22 provides a component strength factor, R_{po}, that accounts for the reserve strength in the component, but also requires calculation of a ductility reduction factor, R_μ, that accounts for ductility and overstrength in the supporting structure. Therefore, for purposes of determining the overall demand reduction predicted by each method, it may be more reasonable to compare R_p with the product of R_{po} and R_μ. In this paper, R_μ is calculated to be 2.06 based on AASHTO [2020] values of $R = 5.0$ and $\Omega_o = 1.3$. Table 4 summarizes the calculated ratio of R_p to the product of R_{po} and $R_{\mu,}$ with values greater than 1.0 suggesting that ASCE 7-16 permits greater reduction for the given structure and component.

Table 4. Comparison of ASCE 7-16 and 7-22 component amplification and reduction factors assuming $R\mu = 2.06$

Component	Equivalent ASCE 7 Component	$R_p/(R\mu \times R_{po})$	a_p/R_p	$C_{AR}/(R\mu \times R_{po})$
Pipe	Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2.19	0.28	0.24
Pole	Other flexible architectural components – High deformability elements and attachments	1.13	0.71	0.45

Table 4 suggests that the total reduction in seismic demand is greater in ASCE 7-16 for both archetype components on this bridge. However, while the reduction permitted in ASCE 7-22 is generally smaller, recall that component ductility is also considered in development of the C_{AR} value and that C_{AR} is generally smaller than the corresponding a_p (see Table 1). This means that a direct comparison of R factors may not tell the full story if various sources of demand reduction have been reshuffled from one coefficient to another. In addition, because the demand reduction potential of the supporting structure is explicitly considered in ASCE 7-22, it is not immediately clear whether ASCE 7-16 or ASCE 7-22 generally predicts lower seismic demands (in terms of allowances for ductility, damping, and overstrength) for any given component without consideration of the supporting structure. A comparison of a_p/R_p with $C_{AR}/(R_\mu R_{po})$ in Table 4 shows little change for the pipe (representative of rigid components) and a more notable change for the pole (representative of flexible components) between the two editions of the code.

Nonetheless, it is important to note that structural demand reduction and overstrength factors can be defined in many ways depending on the design method (force vs displacement) and code used, and that the role of bearings, soil structure interaction, and other sources of demand reduction for bridges may warrant inclusion for bridge applications. Given the importance of this factor to driving overall demands and the fact that ASCE 7 and AASHTO may define and calibrate prescriptive values of R differently, bridge designers need to approach this variable with care if using ASCE 7-22.

5.4.1.4 *Component Seismic Demand*

Considering all factors in the prior sections and their interrelations, it is important to determine the total seismic coefficient predicted for the archetype components and supporting bridge, independent of seismicity (i.e., PGA). This coefficient is calculated using Equations (1) and (2) without the $0.4S_{DS}$ factor. Figure 3 compares the total seismic coefficient independent of seismicity for each component and its variation with the period of the bridge structure.

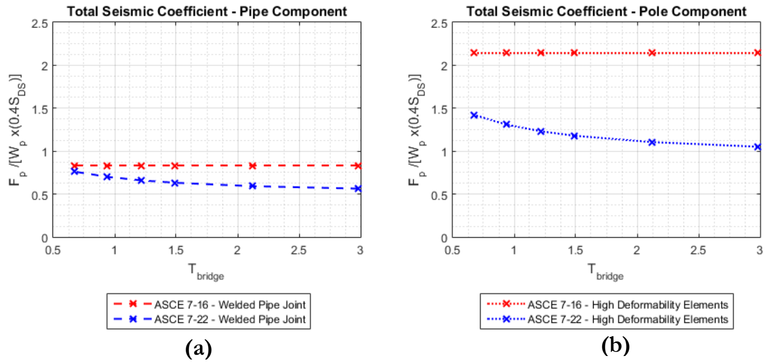
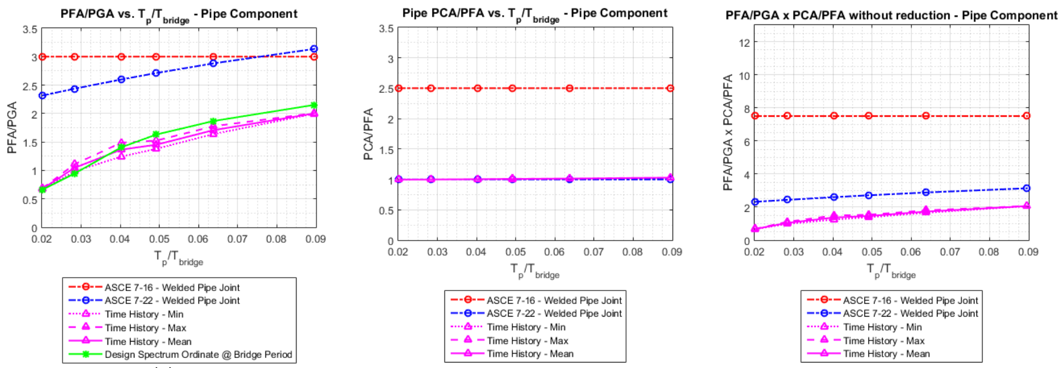


Figure 3. Total seismic coefficients independent of seismicity: (a) Pipe Components, (b) Pole Components

Results in Figure 3 are under the ASCE 7 cap for the seismic design force. Evidently, ASCE 7-22 results in a lower seismic coefficient for both archetype components. A noticeable difference between ASCE 7-16 and 7-22 is that the latter accounts for the impact of the period of the supporting bridge structure on the component demand. As the period of the bridge decreases, ASCE 7-22 equation results in larger seismic demands which is attributed to larger PFA/PGA amplification for short period structures.

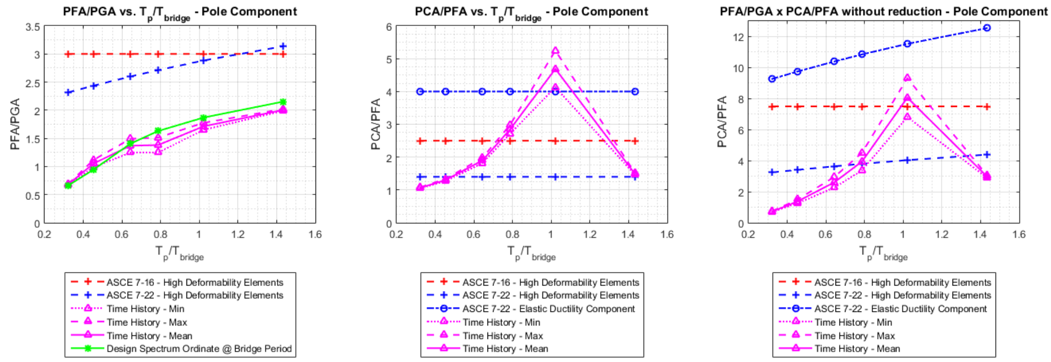
5.4.2 ASCE 7 vs. Time History Analysis

In an attempt compare results from prescriptive code provisions to numerical analyses, a linear time history analysis was performed for each bridge-component system. Accelerations at the deck level (PFA) and at the component level (PCA) were extracted. Figure 4 and Figure 5 compare the amplification factors from the dynamic analysis with those from ASCE 7-16 and ASCE 7-22 for different period ratios. Since the THA is linear, the component reduction factors from ASCE 7 are not considered to better compare the dynamic results. Note that the minimum and maximum THA results correspond to an individual ground motion while the THA mean is the average result.



(a) (b) (c)

Figure 4. Comparison of amplification for pipe components: (a) PFA/PGA, (b) PCA/PFA, (c) PFA/PGA x PCA/PFA



(a) (b) (c)

Figure 5. Comparison of factors for pole components: (a) PFA/PGA, (b) PCA/PFA, (c) PFA/PGA x PCA/PFA

Figure 4a and Figure 5a show that both ASCE 7-16 and 7-22 predict PFA/PGA consistently higher than THA for this bridge. This could be attributed to the code equations being based on regression considering the mean response plus some standard deviation [ATC, 2018], as well as the impact of higher mode effects on multi-story buildings. For comparison, we plotted the spectral ordinate of the target response spectrum to which the time history was scaled, which is a close match to the THA, as would be expected given the simple nature of the bridge bent model. Nonetheless, it is evident that ASCE 7-22 PFA/PGA follows the same trend as THA whereas ASCE 7-16 does not, which suggests that ASCE 7-22 correctly considers the impact of supporting structure period, even if the overall amplification suggested through the z/h = 1.0 assumption appears highly conservative. We note that if the user were to assume a value of z/h less than 1.0 (such as 0.5) or remove the amplification term and replace PGA with a conservative value of the design spectral acceleration for the structure, the code demands would fall significantly, and the analysis results would be a much better fit to the code for a dynamically simple structure such as this.

Figure 4b shows that the PCA/PFA for the pipe from the dynamic analysis closely agrees with ASCE 7-22 but is noticeably lower than ASCE 7-16. Similarly, as shown in Figure 4c, the total amplification predicted by THA is closer to the ASCE 7-22 values but much smaller than ASCE 7-16. It is possible that the relationship between the component amplification and reduction factors plays a role here for this particular component type (note that R_p for pipes under ASCE 7-16 was equal to 9). When considering R_p for ASCE 7-16 and the product of R_{po} and R_{μ} for ASCE 7-22, then the results are nearly identical between the two methods and the THA (if the THA results were to be reduced in accordance with ASCE 7-22).

In contrast, Figures 5b and 5c show that PCA/PFA and total amplification for the pole from the THA exceeds the code predicted values for a wide range of period ratio, T_p/T_{bridge}. The noticeable amplification observed for the pole from THA, in contrast to that observed for the pipe, suggests that a pole is more likely to have fundamental period that overlaps with that of the bridge, and thus is more likely to experience resonance. The large difference in the PCA/PFA values for the pole between THA and ASCE 7-22 may, in part, be attributed to a cap placed in the formulation of ASCE 7-22 PCA/PFA, which was developed with an acceptable probability (10 percent) that the component demand is greater than the code prescribed value within a narrow band of period ratio ($0.85<T_p/T_{bridge}<1.25$). Perhaps more importantly, we have noted that component amplification factors in ASCE 7 are not purely dynamic factors and still consider component damping and ductility; in fact, ASCE 7-22 includes an elastic component ductility category with a higher C_{AR} factor, rather than simply reducing the corresponding R_{po} factor for elastic behaviour. Since our THA is a linear elastic analysis and considered only nominal modal damping (5 percent for all modes), using the elastic component $C_{AR}=4.0$ (as permitted by the commentary of the code) gives a result much closer to the mean peak PCA/PFA of 4.7 determined from the THA (Figure 5b). In contrast, the maximum PCA/PFA provided in ASCE 7-16 is 2.5 (even accounting for $T_p/T_{bridge} = 1$ per the commentary). Thus, this suggests that ASCE 7-22 may provide a higher and more accurate estimate for certain flexible bridge components when compared to ASCE 7-16, so long as the designer appropriately categorizes the component ductility when selecting PCA/PFA value.

6. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

There are no clear bridge-specific guidelines for prescriptive seismic design of bridge-mounted nonstructural components. The use of ASCE 7 for this purpose was evaluated and differences in demands on archetypal bridge components were compared for two editions of the code, 7-16 and 7-22, for an archetype bridge structure. ASCE 7 amplification factors of archetypal bridge components were then compared with a simplified THA of bridge-component systems. The following conclusions can be drawn from this study:

- Overall, the ASCE 7-22 framework for nonstructural component design appears to be a good candidate for application to bridge structures, with *total* design demands results that match or conservatively envelope, and trend more closely with, the analytical results for this example. In addition, it provides a more nuanced framework based in dynamic principals that should allow designers to more transparently apply engineering judgement to reduce conservativism where supported by first principals or sound analysis. ASCE 7-16, in comparison, both over- and underestimated the analytical results for these case studies but may still yield acceptable results for many bridges.
- The primary source of conservativism between ASCE 7 and the analytical results for this case is the structural amplification factor (i.e. PFA/PGA), especially when z/h is, reasonably, assumed to be 1.0 for deck-mounted components. ASCE 7's amplification equation attempts to conservatively envelope floor spectra for multi-story buildings. While this may yield accurate results for relatively flexible bridges or those with appreciable dynamic response of the deck, this may be overconservative for relatively simple bridges dominated by individual pier response that behave like SDOF structures. In these cases, replacing the PGA (i.e. $0.4S_{DS}$) and the amplification factor with a more accurate value of the floor/deck response (such as the design spectral ordinate for the structure), may be appropriate. It is recommended that suitable bounds on period be selected (considering some level of structure overstrength and elastic response) to assure a reliable design, similar to the T_a cap placed on period for force determination in ASCE 7.
- For the examples chosen, ASCE 7-22 component amplification factors, if selected appropriately, seem to yield better agreement with numerical analysis than similar ASCE 7-16 factors. We note, however, that the ASCE 7-22 C_{AR} factors appear to include assumptions about (a) component overstrength and energy dissipation that may link them to the corresponding component reduction factors and (b) acceptable probability of damage or failure of components whose period approaches that of the supporting structure. These assumptions may or may not yield consistent component reliability in light of typical bridge dynamic response and may not be consistent with the expected reliability of bridge-supported utility components and should be reviewed. Use of a larger component importance factor may be appropriate for critical lifelines.

- A key change to ASCE 7-22 compared to 7-16 is the addition of a structure ductility reduction factor, R_μ. While philosophically appropriate to consider, it is unclear how this factor was calibrated for use with typical ASCE 7 lateral system parameters (R and Ω_o) to provide reliable designs, and how potential misapplication of this factor with analytically derived or prescriptively defined (based on other codes like AASHTO or other force- or displacement-based design methodologies) might impact that reliability. When in doubt, use of the minimum structure ductility reduction factor, R_μ, of 1.3 recommended in ASCE 7-22 may be conservative and acceptable.

The proposed use of ASCE 7 as a framework for nonstructural component design for non-building structures is by no means new or novel [Goel 2018]. However, engineers should recognize that although believed to be a good candidate for bridges, ASCE 7-22 non-structural provisions were developed for buildings. Thus, designers need to be mindful of this when applying ASCE 7 or interpreting results, pending clearer guidance to practitioners on reliable application of the framework to bridge structures.

The following are limitations with the current study and recommendations for the future:

- This paper considered just two nonstructural components on a single bridge type, with varying lateral stiffness and subjected to two ground motion records, with the intent of simulating a simple component design validation exercise that a bridge designer might undertake. In the future, the scope of the study should account for more components, bridge configurations and structure types, and ground motions with the goal of better validating the ASCE 7 equations for a wider variety of bridges. Additional parameters that impact dynamic properties such as material and construction method should be considered.
- A detailed study of structural amplification (PFA/PGA) for typical bridge structures is warranted to assess the accuracy of the ASCE 7 equations and, if warranted, provide guidance for obtaining less-conservative results.
- The study did not account for nonlinear behaviour in the supporting structure or component. Future studies should consider nonlinear time history analysis and compare results to ASCE 7 accounting for response modification factors.
- The study presented herein focused on seismic response of bridge-component systems in the transverse direction. For bridge decks, vertical acceleration at mid-span as well as longitudinal seismic forces may be significant. Further studies focusing on vertical accelerations and longitudinal loading need to be performed.
- Additionally, it would be important to compare the seismic demands determined herein to typical wind loads to determine if design is ultimately governed by wind or seismic detailing for certain component types.
- We would encourage future refinements to the ASCE 7 nonstructural design framework to more consistently decouple the component dynamic amplification from the corresponding component reduction factors, where feasible, to improve consistency with numerical methods.

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