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Seismic Design of Piers and Wharves: A Review of Canadian Practice

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ABSTRACT

The publication of ASCE 61-14 "Seismic Design of Piers and Wharves" marked the beginning of standardized seismic design practice of near-shore marine structures in the United States. Since an equivalent design standard does not exist in Canada, marine structural practitioners have sought to incorporate applicable sections of the American standard into Canadian practice. Although a selective adoption of ASCE 61 seems reasonable given the cross-border commonalities in design codes, there are crucial differences in the performance assessment and the calculation of seismic loading on structures between the two countries, including the degree to which inherent overstrength is assumed to exist in new structures along with the associated reduction in seismic demands, as well as the target probability of exceedance of design earthquake events. These differences should be considered when interpreting and applying ASCE 61 requirements for the seismic design of marine structures in Canada so that compliance with minimum Canadian code requirements, objectives, and intent can be realized. This paper provides a brief history of the development of select aspects of seismic design practice in Canada and the US, identifies where major relevant differences in cross-border design practice exist, and provides observations and commentary on the selective adoption of ASCE 61 for the seismic design of piers and wharves in a manner that is consistent with Canadian practice.

Keywords: Seismic, Piers & Wharves, ASCE 61, NBCC, CSA S6.

INTRODUCTION

Although several existing national and international codes, standards, and guidelines cover many aspects of marine structural design, there is no unifying consensus-driven document, published by either a Canadian or American standards association, that provides a set of comprehensive minimum requirements, loads and load combinations, hazard levels, and performance goals for the design of marine structures in North America. Local and specialized codes and design specifications such as POLA [1], POLB [2], UFC [3], and MOTEMS [4] are available in the US, but are of limited scope and application. The American Society of Civil Engineers' (ASCE) Coasts, Oceans, Ports and Rivers Institute (COPRI) is currently developing a nationwide standard to address unique aspects of marine structural design and provide a comprehensive design guidance for piers and wharves in the United States [5]. No such standard is known to be planned or under development in Canada.

In 2014, ASCE-COPRI published ASCE 61-14 "Seismic Design of Piers and Wharves", a limited-scope standard for the design of piers and wharves that addressed the seismic design, analysis, and detailing requirements for new near-shore, piled marine structures. This standard considers structural typology and other unique features commonly found on pier and wharf structures, including batter and slender piled foundations, significant soil-structure interaction, in-water setting, generalized mass concentration at deck level, and the presence of heavy industrial deck equipment, among others [6]. Once this seismic standard was published, and since no equivalent publication existed in Canada, marine structural practitioners took notice and sought to incorporate applicable sections of the new standard into Canadian practice. Although a careful and selective adoption of ASCE 61 is reasonable given the cross-border commonalities in design codes, there are known differences in the seismic design procedure between the two countries, including the degree to which inherent overstrength is assumed to exist in new structures, the associated reduction in seismic demands, and the target probability of exceedance of design earthquake events. Major cross-border differences should be identified, outlined, and understood so that a selective adoption of ASCE 61 can take place in a manner that is consistent with Canadian practice, and which complies with minimum Canadian code requirements, objectives, and intent.

This paper provides a brief history of the development of important aspects of seismic design practice in Canada and the US, outlines some of the ways in which seismic design practice differs across the border, and provides commentary on the selective adoption of ASCE 61 for the seismic design of piers and wharves in Canada. This paper does not discuss at length the technical background of the development of seismic hazard maps by the Geological Survey of Canada (GSC) or the United States Geological Survey (USGS), except where appropriate for context or comparison purposes.

HISTORICAL DEVELOPMENT OF SEISMIC REQUIREMENTS IN THE US AND CANADA

Unites States Hazard/Risk Maps

In 1997 the Building Seismic Safety Council (BSSC) published the National Earthquake Hazards Reduction Program (NEHRP) recommended provisions for seismic regulations for new building structures [7], which adopted newly developed USGS maps and introduced maximum considered earthquake (MCE) ground motions based on both probabilistic and deterministic seismic hazard descriptions. The MCE was defined as "the maximum level of earthquake ground shaking considered as reasonable to design structures to resist" and was based on a 2% probability of exceedance in 50 years (2,475-year return period) capped by a deterministic hazard defined as the "best estimate of ground motion from the maximum magnitude earthquakes on seismic faults with high probabilities of occurrence" [8]. The MCE terminology was adopted "in recognition that ground motion of this probability level is not the most severe motion that could ever [a]ffect the site, but is considered sufficiently improbable that more severe ground motions need not practically be considered. In regions near major active faults, such as coastal California, estimates of ground motion at this probability of exceedance can produce structural demands much larger than has typically been recorded in past earthquakes. Consequently, in these zones, the maximum considered earthquake [was] commonly taken based on conservative estimates of the ground motion from a deterministic event, representing the largest magnitude event that the nearby faults are believed capable of producing" [9].

The adoption of a deterministic cap, in lieu of a country-wide probabilistic approach, was partly based on the observation that "the maximum earthquake for many seismic faults in coastal California is fairly well known and associated with probabilities larger than a 10% probability of exceedance in 50 years" (475-year return period), and that introducing a "radical change in design ground motion in coastal California seems to contradict the general conclusion that the seismic design codes and process [were] providing an adequate level of life safety" [9]. It was then concluded that since "the level of seismic safety using the deterministic earthquake approach would be approximately equivalent to that associated with a 2% [to] 5% probability of exceedance in 50 years for areas outside coastal California", this probability of exceedance for designs outside of coastal California was acceptable to handle the disparity in seismic margin issues [8]. The regions where the deterministic bounds limited the ground motions included California, western Nevada, coastal Oregon and Washington, and parts of Alaska and Hawaii [10].

At the time the 1997 NEHRP Provisions were published, building code provisions in the US considered a lower seismic hazard design level with a 10% probability of exceedance in 50 years (475-year return period) [11]. Shortly thereafter, in its 1998 update, ASCE 7 incorporated seismic hazard levels based on the 1997 NEHRP Provisions, increasing the return period for the seismic design of new building structures from 475 years to 2,475 years.

Canada Hazard Maps

Similar changes in the development of seismic hazard levels were occurring in Canada in the late 1990s and early 2000s, when the Canadian National Committee on Earthquake Engineering (CANCEE) proposed updating the probability of exceedance for the design of new buildings from 10% in 50 years (475-year return period) prescribed in the 1995 edition of the National Building Code of Canada (NBCC) to 2% in 50 years (2,475-year return period), ultimately prescribed in the 2005 edition of NBCC. Although indirect references to the 1997 NEHRP Provisions can be found in the background literature, suggesting a plausible influence of US standards development in Canadian practice, there also appears to be an independent, albeit equivalent rationale that justifies the changes made to the seismic design requirements in NBCC. CANCEE recognized that "the contribution of various sources of conservatism (e.g., overstrength) in the design and construction process leads to a much lower probability that structural failure or collapse will occur due to strong seismic ground motion", and that "seismic hazard calculations at different probabilities of exceedance […] demonstrated that the slopes of the hazard curve vary considerably in different parts of the country". Therefore, in order "to provide a more uniform margin of collapse it [was] necessary to specify seismic hazard at a lower probability of exceedance, i.e., one that is much nearer to the probability of failure or collapse" [12]. As explained by J. Adams and S. Halchuk [13], the performance of pre-2005 provisions, based on a 10%-in-50-year probability of exceedance, "as deduced from global engineering experience with buildings in earthquakes, appears much better than the probability level used would suggest", and "the 2 [% in] 50 year probability level represents the approximate structural failure rate deemed acceptable". Based on this rationale, and understanding "that applying a national, or even regional multiplicative factor" would "not reproduce lower probability hazard values reliably" due to the variability of the slopes of the hazard curves across the country, "CANCEE concluded the direct calculation of seismic hazard at the probability level most appropriate for design is necessary", and that the 2%-in-50-year "seismic hazard values should be used to help achieve an improved, uniform level of safety". Similar to the work being done in the US at the time, the GSC chose to provide a deterministic cap associated with Cascadia earthquake ground motions that affected coastal British Columbia, the occurrence of which was deemed to have an estimated probability of exceedance of about 10% in 50 years.

Cross-Border Comparison of Hazard Maps

Despite differences in the approach taken in the development of hazard maps between Canada and the US, cross-border comparisons, carried out at the time when significant changes were instituted in the late 1990s and early 2000s, found significant similarities and concluded that the overall distribution of seismic hazard was "rather similar" in the border regions [13-15]. Although major changes were subsequently made to the hazard maps, including revisions to the probabilistic treatment of Cascadia and other fault sources in Canada in lieu of the deterministic model [16] and the introduction of risk-targeted maps in the US [17], "the similarity in level and pattern across the Canada-US border [remained] generally good" [18].

The current edition of the seismic maps presented in ASCE 7-22 [19] represent a 1%-in-50-year collapse risk target (i.e. risk coefficient maps), while in Canada the current edition of the seismic maps represents a 2%-in-50-year hazard level (i.e. probability of ground motion exceedance) [20]. The change from hazard level to collapse risk target approach currently used in the US maps was first introduced in the 2010 edition of ASCE 7, along with other changes in provisions related to the deterministic ground motions magnitude assessment and the evaluation of liquefaction potential. The introduction of risk coefficient maps aimed to provide a uniform collapse probability of structures across the country, since the previous hazard maps were deemed to result in structures with different probabilities of failure due to the variations in the factor of safety against collapse relative to the ground motion for which the structures were designed [21]. It should be noted that a uniform hazard of 2% in 50 years ground motion exceedance and deterministic ground motions (84th percentile) are still implicit in the current American risk coefficient maps [22]. While Canada also explored the utility of a risk-target approach, the continued use of uniform ground motion exceedance probabilities was deemed to have "little-to-no impact on life safety" and a risk-target approach has not been adopted to date [23].

Design Base Shear Comparison

When ASCE 7-98 increased the return period for the seismic design of new building structures from 475 years to 2,475 years, the calculation of seismic demands on structures was not to consider the full set of seismic loads produced by the MCE. Instead, base shear demands, calculated using the new seismic hazard maps, were factored by 2/3 in design. This reduction in seismic demand was consistent with arguments laid forward in the 1997 NEHRP Provisions, which stated that the "collective opinion of the SDPG [Seismic Design Procedures Group] was that the seismic margin contained in the Provisions provides, as a minimum, a margin of about 1.5 times the design earthquake motions. In other words, if a structure experiences a level of ground motion 1.5 times the design level, the structure should have a low likelihood of collapse" [8]. As such, the 2/3 reduction factor, which corresponds to the "reciprocal of 1.5" […] "is agreed to be the seismic margin built into structures designed by […] older editions of the NEHRP provisions" [24].

The introduction of a 2/3 reduction factor was a noteworthy divergence between the US and the Canadian building code approach to design, which until then was deemed to result in a "reasonable match of design base shear" magnitudes for structural systems having a high ductility capacity after accounting for their different treatment of ductility and "calibration" factors [25]. The introduction of the reduction factor would ultimately contribute to differences in design base shear between the codes despite seemingly comparable performance objectives and effective inter-story drift limits. Until then, aside from format, the design base shear in the US and Canada was deemed to not have varied too much over the past four decades [26]. The GSC noted at the time that although the Canadian adoption of a new ground motion spectra corresponding to an exceedance probability of 2 % in 50 years was "consistent with recent parallel developments" by the USGS, "the way that the spectra are used for Canadian design [would] differ from the approach in the U.S., which is to design to two thirds of the 2% in 50 year values" [13] [27].

Notable cross-border differences in seismic demands have been observed by practitioners and researchers following the changes in hazard levels introduced in the late 90s, for which comparisons have consistently shown lower seismic demands in US building codes compared to Canadian building codes. Published works for cross-border comparison have mostly addressed the design of ductile systems, and while some authors attribute the differences in design to the degree of ductility allowed by the codes [28], other authors correctly attribute a portion of the "markedly lower" seismic demands in the US to the "2/3 factor applied to the […] spectral ordinates to obtain the design spectrum" [29].

SEISMIC DESIGN OF PIERS AND WHARVES IN THE US AND CANADA

Seismic Design of Piers and Wharves in the US

The first edition of ASCE's "Seismic Design of Piers and Wharves" (ASCE 61) was published in 2014 "to provide minimum requirements for the seismic design of pile-supported piers and wharves". In this standard, performance objectives are provided for 'high', 'moderate', and 'low' design classification of structures, for which associated minimum seismic hazard levels are assigned. The hazard levels for which explicit evaluation is required include the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE). Specific probabilities of exceedances are specified for the CLE and OLE, which vary from 50% in 50 years (72-year return period) to 10% in 50 years (475-year return period) depending on the design classification. The DE hazard level, however, is defined as the "Design Earthquake per ASCE 7 (2005)", which corresponds to a 2,475-year event (or the appropriate deterministic cap) with the resulting ground motions modified by the 2/3 reduction factor used in ASCE 7. It's worth noting that the explicit adoption of the DE hazard levels as those specified in ASCE 7 is to ensure a consistent 'life safety' design approach that aligns with applicable local building codes and that "as a minimum, the criteria for seismic design of piers and wharves are no less stringent than the minimum standards considered for ordinary buildings" [6].

The seismic hazard used in ASCE 61-14 deviated from that provided in the 2010 edition of ASCE 7, which is based on 2009 NEHRP Provisions. Some of the significant changes that had been made to the ground motion maps from ASCE 7-05 to ASCE 7-10 at the time of ASCE 61's publication included changes to zone source modeling and attenuation relationships, uniform hazard maps being replaced by risk-targeted maps, and deterministic ground motions changed from median to 84th percentile estimates. In ASCE 7-10, the uniform-hazard ground motion was replaced with a risk-targeted ground motion by switching from a 2% in 50-year hazard level to a 1% in 50-year collapse risk target. The risk-targeted ground motions involve the integration of a fragility curve for a generic building structure with probabilistic ground motions to determine the risk of collapse. The risk-targeted maximum considered earthquake (MCE) ground motion is designated MCER ground motion. In addition, the deterministic ground motion caps were changed from 150% of median ground motions to 84th percentile ground motions, which are approximately 180% of median ground motions. Ghosh [21] describes the consequences of these changes as a "slight increase or decrease in design ground motions, on average" across the country, estimated to be 10% for the western United States (with notable exceptions such as more than 40% increase in San Bernardino, California and more than 20% decrease in San Diego, California) and less than 15% change almost everywhere else in the 48 contiguous states. Ghosh also notes that the changes from ASCE 7-05 to ASCE 7-10 included the requirement to carry out the "evaluation of liquefaction potential for maximum considered earthquake [MCE] ground motions, rather than design earthquake [DE] ground motions", although this assertion appears to be disputed by commentary found in ASCE 7-16.

The change from uniform to risk-targeted ground motions, as well as the changes to the deterministic caps, occurred too late into the development process of ASCE 61 for incorporation. The committee did not have sufficient time to understand and explore the implications and appropriateness of the hazard changes for inclusion into a new standard, so the older uniform hazards are used by a direct reference to ASCE 7-05. The reason for specifying the 2005 edition of ASCE 7 was ultimately cited as: "ASCE 7-10 has adopted significant changes from ASCE 7-05, especially in the definition of ground motions, and at the time of writing of this standard, ASCE 7-10 has not been adopted for use by any regulatory authorities and has not been used for design of marine structures. Thus, the committee has decided that it would be appropriate to reference the version of the document currently being used, rather than the newer edition" [6].

As explained further below, the next edition of ASCE 61 plans to step away from referencing ASCE 7 for the DE event and will instead reference the American Association of Highway and Transportation Officials' (AASHTO) design earthquake.

Seismic Design of Piers and Wharves in Canada

Without the benefit of a specialized seismic design standard for piers and wharves, the seismic design of marine structures in Canada has historically relied on provisions by NBCC or the Canadian Highway Bridge Design Code (CSA S6). During the early 90s and until the 2005 edition of NBCC, there was generally good alignment in the seismic design hazard levels specified for ordinary building structures and typical bridges in Canada, with both NBCC and CSA S6 adopting a 475-year probabilistic event (10% probability of exceedance in 50 years) [30-34]. From 2005 to 2014 the Canadian bridge code continued to specify design hazard levels of this magnitude while, in contrast, NBCC introduced a more stringent 2,475-year design event for ordinary structures [35-36]. This discrepancy in minimum design requirements made the bridge code the go-to seismic design standard for marine structures in Canada, as it offered significant economic advantages to facility owners while still maintaining compliance with Canadian practice. This choice of code was also supported by the generally correct but low-resolution argument that 'marine structures are more like bridges than like buildings'. In 2014 the Canadian Standard Association (CSA) published the 11th edition of the bridge design code, which brought back alignment with NBCC by raising the seismic design hazard level to a 2,475-year design event with an associated minimum performance goal of life safety for typical bridges [37]. Though seismic design practice has continued to evolve in Canada and the GSC has introduced some changes in the treatment of probabilistic and deterministic ground motions during the past decade, this life-safety design hazard level has remained unchanged through the current edition of the bridge code and NBCC [38-39].

With the introduction of ASCE 61-14 in the United States, individualized attempts by practitioners, owners, and jurisdictions have sought to adopt relevant design requirements of the American standard into Canadian practice in a manner that is consistent with the minimum requirements and intent of the Canadian codes. The manner in which ASCE 61-14 is currently adopted in Canada seems to vary by project, region, facility owner, Authority Having Jurisdiction (AHJ), and practitioners' preference.

APPLICATION OF ASCE 61 IN CANADIAN PRACTICE

Common Misconceptions

Although ASCE 61-14 aims to provide a design approach that aligns with local building codes in the United States, there are several aspects of this standard that apply specifically to piers and wharves, including considerations for in-water setting, significant influence of soil-structure interaction, unique nature of marine structural typologies, and the presence of large mobile or fixed equipment such as cranes, shiploaders, and marine loading arms. In recognition of ASCE 61's intent, and accounting for known cross-border design practice differences, the adoption of relevant ASCE 61 provisions for the seismic design of marine structures in Canada should consider the following common misconceptions:

- *Misconception No. 1: A 2/3 reduction factor applied to the GSC ground motions is appropriate for the design of marine structures in Canada because piers and wharves need not adhere to the same standards as building structures (on low occupancy merits).* As explained above, the intent of ASCE 61 is to provide standards that are no less stringent than the minimum standards considered for ordinary buildings. In the US, the 2/3 reduction factor is used in the design of building structures and is therefore also applied to the design of piers and wharves. In Canada, the minimum standards for ordinary buildings do not consider a 2/3 reduction factor and therefore it is ill advised to apply a reduction factor in the design of marine structures if compliance with the Canadian code intent is sought.
- *Misconception No. 2: The 2/3 reduction factor, applied to the 2475-year ground motions, produces equivalent design values as those corresponding to a 1,000-year seismic event; as such, these definitions can be used interchangeably*. While it is true that for many sites across North America a 2/3 reduction to the MCE ground motions will result in design values similar to those produced by a 1,000-year design event, there are good reasons to avoid this simplified characterization. In ASCE 7, the reduction factor is intended to be applied to the 2,475-year ground motions for structural design but not for evaluation of liquefaction potential. Characterizing the design earthquake event simply as one with 1,000-year return period may omit this important distinction when ASCE 61 adoption is referenced in Canadian designs.
- *Misconception No. 3: ASCE 61-14 defines the design earthquake hazard level as that specified in the 2005 edition of ASCE 7, and therefore the standard permits the liquefaction potential to be evaluated at the reduced ground motions (i.e. based on 2/3 of MCE ground motions).* While it is true that the language in ASCE 7-05 specifies liquefaction potential to be evaluated at the DE level (i.e. including a 2/3 factor) [40], ASCE clarified that "there was ambiguity in the previous requirement in ASCE 7-05 as to whether the liquefaction potential should be evaluated for the MCE or the design earthquake" and that "beginning with ASCE 7-02, it has been the intent that liquefaction potential be evaluated at the MCE ground motion levels". It is also acknowledged that "by multiplying peak ground acceleration by a factor of 2/3, liquefaction would be assessed at an effective return period or probability of exceedance different than that for the MCE" [22], which is not the intent of the standard. Given these clarifications, it would seem imprudent to take the ASCE 7-05 requirements at face value and apply a 2/3 reduction to the 2,475 ground motions used for liquefaction assessments in Canada.

It's worth highlighting that NBCC also acknowledges that some reserve strength against collapse exists in building structures designed in accordance with its provisions. NBCC reduces the seismic design force by use of the R_o term in the base shear equation (in combination with the ductility-related force modification factor R_d). It does so to explicitly account for this over-strength in lieu of the pre-2005 "calibration factor" U, sometimes called the "level of protection experience factor", which was used to calibrate code seismic design forces to historical levels used in older editions of the code [41-42]. Therefore, the introduction of the US 2/3 reduction factor, applied at the front-end of the design ground motions obtained from CGS maps, may risk double counting on the overstrength considerations already built in Canadian design practice.

The Future of ASCE 61

The second edition of ASCE 61 is planned for publication over the next few years. A major change currently proposed is to replace the current DE hazard level of 2/3 of MCE as per ASCE 7-05 (i.e. 2/3 of 2,475-year event) by the hazard level specified in AASHTO, which is based on a 5% probability of exceedance over a 50-year period (975-year return period) [43]. If this change is adopted by ASCE 61, Canadian practitioners are encouraged to consider the following:

- The seismic provisions of the Canadian bridge code that were developed in the late 90s retained the basic format of AASHTO LRFD [44-45]. As such, CSA S6 and AASHTO LRFD shared many similarities in their seismic design approach prior to 2009, including the 475-year return period of the design seismic event, the formulation of the elastic seismic response coefficient, and the ultimately effective treatment of ductility considerations and importance factors to estimate design base shears [36] [46-47]. Many of these similarities came to an end with the publication of AASHTO's Guide Specifications for seismic design [48], in which the return period of the seismic design event was updated to 1,000 years (5% in 50 years probability of exceedance) [49]. Further divergence between the codes took place in 2014 when CSA S6-14 adopted a 2,475-year design event (2% in 50 years probability of exceedance) to better align with Canada's National Building Code provisions [50].
- In the early 2000s, the National Cooperative Highway Research Program (NCHRP), which developed the recommended specifications for the seismic design of highway bridges intended to be integrated into the AASHTO LRFD Bridge Design Specifications, recommended that the seismic design of highway bridges consider a 2,475-year design event (i.e. 2% probability of exceedance in 50 years) via Project NCHRP 12-49. They also recommended the omission of the 2/3 reduction factor applied to the ASCE 7 MCE ground motions to "more properly accommodate the design displacements associated with the MCE event. Displacements are much more important in bridge design than building design, since displacements govern the determination of required pier and in-span hinge seat widths, and are thus critically important in preventing collapse" [51-52].
- Following the review of the recommended LRFD seismic guidelines published by project NCHRP 12-49, AASHTO Highway Subcommittee on Bridges and Structures' seismic design technical committee (T-3) established project NCHRP 20-07 Task 193, which produced Task 6 Report "updating" the recommended LRFD seismic guidelines. This committee was tasked specifically with the "selection of a return period for design less than 2,500 years", and produced a report that presented a "justification for the 1,000-year return period (i.e. 5% probability of exceedance in 50 years)" for the seismic design of highway bridges [53-54]. Little commentary can be found on the simultaneous change of target performance at the design event, which changed from repairable damage performance (in the earlier version of AASHTO) to life safety performance (in the proposed revisions), implying a rather gradual change in the overall code design requirements when the 1,000-year return period event was introduced.
- The justification for a reduction in the recommended hazard level from 2% in 50 years (proposed by NCHRP 12-49) to 5% in 50 years (eventually adopted by AASHTO LRFD) included comparisons of spectral accelerations for different sites across the continental US with consideration of maximum deterministic and probabilistic ground motions magnitudes, as well as comparison of 'design' ground motion magnitudes (i.e. MCE event with a 2/3 reduction factor) between these sites. These comparisons, used to justify the adoption of a 1,000-year seismic design event for bridges across the US [55], would not be necessarily applicable to Canadian practice given the full probabilistic treatment of ground motions used in the CGS hazard maps (i.e. the exclusion of a deterministic approach) and, more importantly, Canada's exclusion of a 2/3 reduction factor applied to the 2%-in-50-years ground motions in its design approach.
- As per NBCC, CSA S6 specifies a minimum life safety performance at the 2,475-year return period event for the design of bridges in Canada, albeit with variable definitions of life safety performance, which, in the bridge code, expands well into the near collapse state of the structure. In any case, even if ASCE 61 adopts a 975-year design event in accordance with bridge design practice in the US, compliance with Canadian practice would still necessitate piers and wharves in Canada be designed using the 2,475-year event to satisfy minimum national standards.

ASCE 61-14 and CSA S6-19

There are notable similarities in the seismic design philosophy presented in ASCE 61-14 and CSA S6-19, which supports the argument for a partial and careful implementation of select ASCE 61 provisions in Canadian designs. A key similarity is the adoption of a performance-based design philosophy, for which performance requirements are assigned and explicitly evaluated at different hazard levels, in accordance with the structure's classification. This design philosophy is not shared with NBCC. Although the minimum target performance for ASCE 61-14 and CSA S6-19 is the same: life safety at the 2,475-year event (regardless of the structural classification), compatibility of performance requirements for all classifications and hazard levels is not self-evident. For example, as shown in Table 1, the highest ASCE 61 classification-level requires repairable damage performance at the 475-year event, which matches the descriptive performance requirement for S6's lowest classification of 'other bridges' for the same hazard level. However, ASCE 61 also specifies explicit performance evaluation at a third, lower hazard corresponding to a 72-year event, while in CSA S6 no such explicit evaluation is mandated.

Table 1. ASCE 61-14 and CSA S6-19 Select Performance Requirements

Hazard Level	ASCE 61-14 High Classification	CSA S6-19 Other Bridges
2% in 50 years (2,475-year Return Period)*	Life Safety Protection	Life Safety
10% in 50 years (475-year Return Period)	Controlled and Repairable Damage	Service Limited (repairable)
50% in 50 years (72-year Return Period)	Minimal Damage	-

* Base shear multiplied by 2/3 in ASCE 61-14

Based on a comparative assessment, one may conclude that while reasonable alignment exists between ASCE 61's 'high' classification and CSA S6's 'other bridges', no such alignment exists for any other design classification across the publications. Compared to 'other bridges', CSA S6 requires higher performance for 'lifeline' or 'major-route' bridges at intermediate hazard levels of 5% in 50 years and 10% in 50 years. When compared, ASCE 61 intermediate-hazard performance requirements for 'moderate' and 'low' design classifications are well below the minimum standards specified in CSA S6 for all design classifications. ASCE 61-14 provides some rationale on the selection of minimum performance requirements at intermediate hazard levels for piers and wharves that include differences in expected occupancy compared to building structures, as well as relatively low life safety risks associated with private versus public access considerations. This kind of rationale may be invoked to justify apparent discrepancies with intermediate design requirements between ASCE 61 and CSA S6, including those potentially attributable to differences in specified design life, provided Canadian minimum life safety requirements are upheld in the design of piers and wharves.

Other parallels between ASCE 61 and CSA S6 include the recognition of distinctive structural typologies, specified methods of analysis in accordance with structural configuration, distinctive requirements for force-based design and elastic analysis methods, differences in specified materials of construction, and detailing requirements, all of which affect the intended performance of the structures. Although some similarities exist in the minimum requirements for design approach, methods of analysis, and detailing considerations between ASCE 61 and CSA S6, partly because ASCE 61 borrows from American bridge design specifications such as Caltrans [56-57] and AASHTO [46], it should be of no surprise that some differences should exist between bridge design code requirements and those considered appropriate for the design of piers and wharves. One notable example is the design approach for structures with batter/raked piles, which is a very common structural configuration found in piers and wharves. ASCE 61 requires that, structures with batter piles for which ductile behaviour is desired, the ductile performance must be demonstrated by analysis using non-linear push-over methods, while the simplified linear analysis, 'R'-factor approach is not permitted. An example where ASCE 61 will enhance CSA S6 is the explicit joint detailing requirements for cracked and uncracked joints, including longitudinal, spiral, and shear reinforcement, which can be considered complementary to the Canadian bridge code requirements. Other examples where ASCE 61 will provide more specific considerations on methods of analysis and detailing of connections include the moment-curvature analysis methods permitted for non-linear analyses, which depend on whether the pile-to-deck connection is a steel-pipe pile connection (embedded pile, concrete plug, isolated shell, welded embed, or welded dowel), a pre-stressed concrete pile connection (pile buildup, extended strand, embedded pile, dowelled, hollow dowelled, external confinement, or isolated interface). Minimum requirements for specified materials of construction may be easier to reconcile since there is little cross-border variability when it comes to concrete and steel reinforcement, and equivalent classes and grades have been recognized for many years and can be more easily substituted. Steel pipe pile grades and fabrication specifications is perhaps an area where further discussion may be warranted, as hollow structural shapes are generally not used for piling in marine structures, and fabrication guidance is often sough in standards from the offshore industry, such as those published by the American Petroleum Institute (API), which introduces further considerations that are beyond the scope of this paper. Overall, given the wide array of structural configurations commonly found in piers and wharves, apparent inconsistencies between the American standards and Canadian codes, such as those identified above, would need to be resolved and agreed upon between the engineer and the facility owner on a case-by-case basis using engineering judgment.

CONCLUSIONS

Seismic design codes in Canada and the US shared considerable similarities in the early 90s. Subsequent editions, released in the 2000s and thereafter, introduced code changes that resulted in notable divergence in cross-border seismic design practice. The main differences, which developed between NBCC and ASCE 7 as well as CSA S6 and AASHTO, centered around the estimation of base shear magnitudes and the effective design-event return period specified for structural design.

In 2014 ASCE-COPRI published ASCE 61-14, a limited-scope standard for the design of piers and wharves that addressed the seismic design, analysis, and detailing requirements for new near-shore, piled marine structures. Without the benefit of an equivalent Canadian seismic design standard. individualized attempts by practitioners, owners, and jurisdictions have sought to adopt relevant design requirements of the American standard into Canadian practice in a manner that is consistent with the minimum requirements and intent of the Canadian codes.

It is possible to lay out a reasonable argument for the design of piers and wharves in Canada using select aspects of ASCE 61, provided that, as a minimum, the following is considered:

- The magnitude of the design ground motions and the return period of the design event at which life safety is evaluated should be consistent with Canadian codes (i.e. 2,475-year return period without the use of a 2/3 reduction factor). This consideration applies for both structural design and geotechnical design (i.e. liquefaction potential evaluation).
- When supplemented by ASCE 61 for seismic considerations, it may be preferable to use CSA S6 rather than NBCC as the ‘base’ design code for marine structures in Canada. The adoption of CSA S6 for marine structural design has already been common industry practice for years, but specifically in the context of seismic design, it is useful that the performance-based design philosophy of CSA S6 is also shared by ASCE 61.
- While there is reasonable alignment in performance targets for ASCE 61’s ‘high’ classification and CSA S6’s ‘other bridges’, owners and practitioners are encouraged to seek early agreement with the AHJ on major seismic design aspects intended to be superseded in CSA S6 and adopted from ASCE 61. These aspects include the intended ASCE 61 design classification for the structures, along with intermediate hazard levels to be evaluated and their associated performance goals, highlighting any major differences with the otherwise expected performance goals that would normally apply to bridges of equivalent CSA S6 classification.
- The basis of design should also highlight, if applicable, major differences in the design approach and methods of analysis adopted from ASCE 61 and those normally required for bridge design in accordance with CSA S6, acknowledging that it is preferred for methods of analysis, material and detailing requirements, and target performance and strain limits to be adopted in their entirety from a single source. Inevitably, equivalent material grades and specifications may be needed for Canadian projects for compliance with national standards, and these potential inconsistencies would need to be resolved and agreed upon on a case-by-case basis using engineering judgment.

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