
# Reinforced Concrete Substructure Bridge Seismic Performance for Mainshock-Aftershock Scenarios – Review of Findings and State of the Art

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## ABSTRACT

Previous earthquakes such as Northridge (1994), Chi-Chi (1999), Tohoku (2011), have shown that large earthquake mainshocks are followed by several aftershocks. Most bridge codes simply ignore the aftershock effects by designing to a peak engineering demand parameter such as spectral acceleration. Past structural performance under large earthquakes has been unsatisfactory especially when aftershocks have occurred, e.g. Christchurch (2010, 2011). With the Owners' need to quantify bridge performance under several levels of seismicity, performance-based design is often the required design methodology. Unlike other codes, the current Canadian Highway Bridge Design Code, CAN/CSA-S6-14 (S6-14) requires consideration of aftershock effects. S6-14 requires seismic capacity retention after the mainshock along with provisions to have full capacity restored by repairs. The term seismic capacity in S6-14 is ambiguous, i.e. it is unclear if S6-14 refers to force capacity or displacement capacity. The BC Ministry of Transportation and Infrastructure Supplement to S6-14 has deleted the requirements for aftershock performance demonstration. This is primarily due to the lack of generally accepted methodologies for such assessment. Some research has accounted for earthquake duration and near-field effects, while separate research has been carried out on performance and resilience of reinforced concrete bridges under mainshock-aftershock scenarios. Other researchers have looked at dynamic instability of structures; however, these investigations have mostly focused on building systems using Incremental Dynamic Analysis based solely on comparisons of a single seismic intensity measure. This paper presents a review of findings and state-of-the-art related to mainshock-aftershock performance of reinforced concrete substructure bridges. The review has identified a lack of unification and the absence of comprehensive studies to quantify bridge performance under the mainshock-aftershock effects. It is recommended that systematic and comprehensive studies be carried out to quantify mainshock-aftershock performance and simplified analysis and design recommendations provided to advance the state-of-practice for safer and more resilient bridges.

Keywords: performance-based design, mainshock, aftershocks, seismic capacity retention, reinforced concrete bridges

## INTRODUCTION

Previous earthquakes such as Northridge (1994), Chi-Chi (1999), Chile (2010), Tohoku (2011), Nepal (2015) have shown that large earthquake mainshocks are followed by several aftershocks. Most seismic codes all but ignore the effects of aftershocks by designing to a peak engineering demand parameter such as spectral acceleration or displacement. Structural performance under many past large earthquakes has been unsatisfactory especially when aftershocks have occurred, e.g. Christchurch (2010, 2011). With analytical tool advancements and the Owners' need to quantify bridge performance under several levels of seismicity, performance-based design is often the required design methodology. The current Canadian Highway Bridge Design Code, CAN/CSA-S6-14 (S6-14) [1] has incorporated the performance-based design as the default methodology for all bridge designs. S6-14 has the following requirements for aftershock performance:

- For Service-Limited and Damage-Repairable, the structure needs to retain 90% of its seismic capacity and have full capacity restored by repairs.

- For Service-Disruption and Damage-Extensive, the structure needs to retain 80% of its seismic capacity and have full capacity restored by repairs.

The above requirements are in addition to specific material strain limits for each performance level. The meaning of the term seismic capacity used in S6-14 is unclear, i.e. is S6-14 referring to force capacity or displacement capacity for aftershock performance compliance. In the absence of any guidelines and until more studies are carried out, one simple interpretation of capacity retention to satisfy the code requirement has been provided by Khan et al. [2] as shown in Figure 1 below:

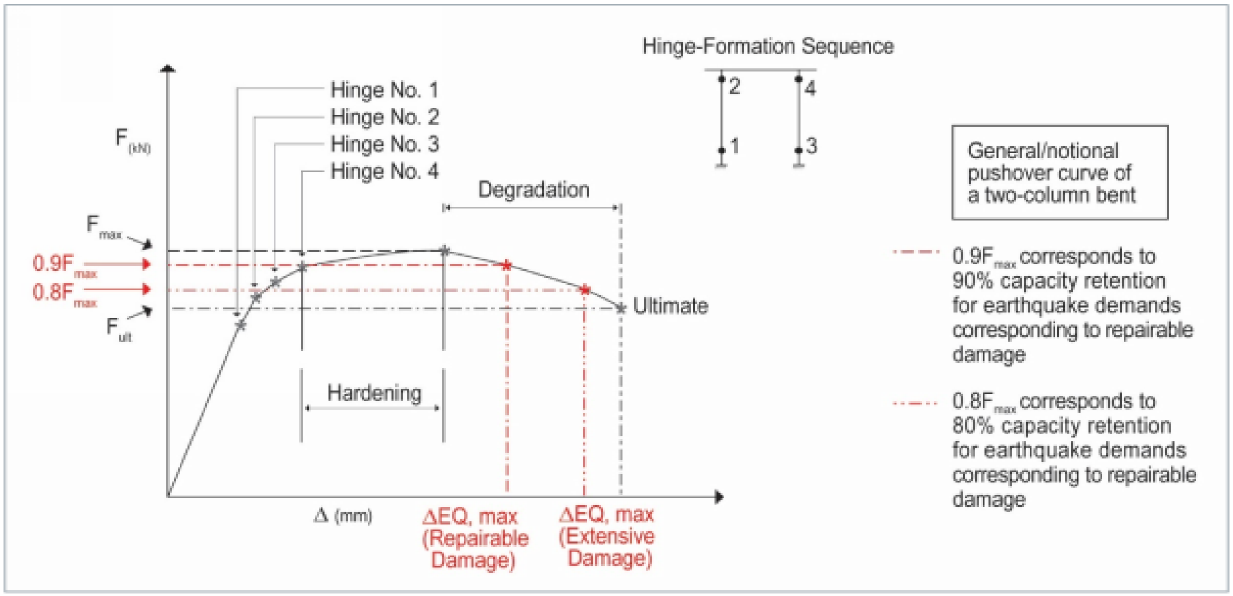


*Figure 1. Aftershock capacity demonstration using inelastic static pushover analysis*

It should be noted that the BC Ministry of Transportation and Infrastructure Supplement to S6-14 (the Supplement) [3] has currently deleted the requirements for aftershock performance demonstration. However, this is due to the lack of generally accepted methodologies for such assessment. The Supplement commentary suggests that elements designed to S6-14 requirements will retain the full capacity after the design event and can sustain multiple additional cycles of seismic loading. However, upon scrutiny, this statement does not appear to be based on a comprehensive study or testing of modern details subjected to dynamic loads. The veracity of the assumptions in the above statement is doubtful as structural and site periods, earthquake duration, near field effects, seismic energy and number of cycles, available ductility and hysteretic behaviour etc. can have vastly different effects on a structure. In addition, non-uniform stiffness and strength attributes are also critical as these are often encountered in bridges and lead to inelastic demand concentrations in certain locations. Well-detailed structures damaged during mainshocks may therefore face significant risk during aftershocks. It is likely due to the recognition of these variables that the upcoming version of CAN/CSA-S6-19 (S6-19) [4] will require consideration of the potential effects of aftershocks on the performance levels, including consideration of the post-earthquake risks inherent in the expected damage states for the chosen lateral load-resisting system. However, as with the current S6-14, the future S6-19 seems to offer no guidance on how to account for aftershock effects. S6-19 has deleted the specific capacity retention requirements in S6-14, making the question even more open-ended for the future designs.

We note that some research has accounted for earthquake duration, near-field effects, etc. while some limited research has been carried out on performance of low height reinforced concrete bridges under mainshock-aftershock scenarios. Limited research has also been carried out on the fragility, risk, and resilience of bridges following aftershocks. Mainshock-aftershock ground motions features have been studied in the context of buildings; however, this knowledge should be directly transferable to bridge performance assessments. Although researchers have also looked at collapse in terms of dynamic instability of structures, these investigations have mostly been focused on SDOF structures and mostly building systems using Incremental Dynamic Analysis (IDA) based solely on comparisons of a seismic intensity measure. This approach generally ignores the reasons record-to-record variability, nature of seismic sources (see FEMA P440A [5]), etc.

This paper presents the findings and state-of-the-art review of research work related to the issue of mainshock-aftershock performance of reinforced concrete substructure bridges. In doing so, appropriate research used in buildings that may be of direct applicability to reinforced concrete substructure bridges has also been reviewed and discussed. The goal of the review is to recognize the strengths of the current research as well as the gaps in our current understanding of mainshock-aftershock effects on reinforced concrete substructure bridges. This review will help guide a future comprehensive study to quantify reinforced concrete substructure bridge performance designed and detailed according to the latest version of CAN/CSA-S6.

## IMPORTANT FACTORS FOR QUANTIFYING STRUCTURAL SEISMIC PERFORMANCE

The two broad categories of factors influencing structural seismic performance can be categorized as seismicity and ground motion features, and, structural design and detailing. Seismicity related factors include but are not limited to, the energy, frequency, duration, polarity, etc. of a given earthquake record or a mainshock-aftershock scenario. Similarly, near field and far field seismic records have distinctly different features. Mainshock-aftershock ground motion features are also of particular interest as these tend to influence structural performance. On the other hand, structural period, strength and stiffness, available ductility, hysteretic behaviour, irregularity, redundancy, etc. are some of the critical factors related to structural design and detailing. It is noted that the structural performance is a function of both seismicity and the structural design and detailing attributes. A summary of some of the aforementioned factors and their effects in the context of mainshock-aftershock scenarios is first described as follows:

### Ground motion period:

Mainshock-aftershock ground motion features tend to play a critical role in structural performance. Ruiz-Garcia [6, 7] analyzed several mainshock-aftershock sequences from the 2010-2011 Christchurch earthquake along with many recorded time histories from the PEER NGA WEST (2011) and the Mexican Database of Strong Motions (1999). It was found that the predominant ground period, which is an indicator of the ground motion frequency content, was consistently longer for the mainshocks in comparison to their corresponding largest aftershocks. In other words, the aftershock ground motions are richer in higher frequencies than the mainshock ground motions. In addition, the statistical correlation between strong motion duration of mainshock and aftershock ground motions was found to be weak. It was therefore concluded that there is little evidence to support mainshock-aftershock seismic sequence simulation using the mainshock as the aftershock, i.e. analyzing structural performance using back-to-back mainshocks.

### Seismic sources:

The seismic hazard in coastal British Columbia derives from three distinct sources, i.e. crustal, deep inslab and mega-thrust/interface (Cascadia subduction zone). Tesfamariam and Goda [8] found that when structures with different fundamental periods (2-, 4-, 8-, and 12-story non-ductile reinforced concrete frames) were subjected to ground motions from different sources, their collapse potential varied significantly. In this study, the aftershocks were found to have little impact on longer period structures (4-, 8- and 12-story frames), while showing a worsening of seismic performance for the short period, 2-story structure. As the structural period lengthened, the crustal and inslab events showed a diminishing performance difference, while showing negligible performance difference for the 12-story frame.

### Earthquake direction and aftershock polarity

Hosseinpour and Abdelnaby [9] studied two eight-story reinforced concrete buildings (both regular and irregular). They used a fibre-based distributed plasticity model and robust finite element analysis and found that the mainshock direction affects the total drift demands and could cause high total residual drifts in irregular structures. The damage effect from the mainshock has more significance when the aftershock is applied to an irregular structure. It has been considered previously that the mainshock-aftershock could have different polarity [10]. The study by Hosseinpour and Abdelnaby employed various sequences to ascertain the differences in total drift demands as a result of the change in aftershock direction. The study found that aftershock polarity (direction) can lead to much larger total drift demands in irregular structures especially with sequences containing high peak ground acceleration (PGA) values. However, the effects of aftershock polarity lead to insignificant changes in drift demands for the regular structure.

Similarly, Ruiz-Garcia and Aguilar [11] used the lumped plasticity approach to investigate the effects of aftershock polarity on a four-story steel moment resisting frame employing a post-mainshock Incremental Dynamic Analysis (IDA) [12]. Although carried out for a steel structure, this study also observed significant effects due to the polarity of the aftershock ground motion. It found that the aftershock polarity not only had a strong influence on the aftershock collapse capacity but also the aftershock capacity corresponding to demolition. The study found that when the aftershock acted with negative polarity, the structure began to develop a re-centering response at low intensities; the residual drifts then ended up in the opposite direction as compared to the mainshock response.

### Energy

An adequate and optimal seismic demand model should be independent of the suite of records that are used to calibrate it [13]. However, the normally used elastic spectral quantities are inappropriate for this purpose. Energy-based intensity measures provide alternative means for capturing the ground motion characteristics. The concept of Effective Cyclic Energy (ECE) as a measure of seismic demand by Kalkan and Kunnath [14] has been found to correlate well with peak structural seismic demand for a range of system parameters. The ECE concept was produced with particular focus on near-field ground motions, which often impart sudden and intense energy into the structure that needs to be dissipated within a short period of time. The earthquake damage is then related to the maximum deformation or maximum ductility. Although the ECE concept has not been used in a mainshock-aftershock framework for bridges, this or other energy-based approaches could prove suitable for quantifying mainshock-aftershock performance through correlation with cumulative input and/or dissipated energy demands and capacities. Tesfamariam and Goda [15] used 50 mainshock and 50 mainshock-aftershock scenarios for a 15-story reinforced concrete building in Vancouver, BC and used the energy-based Mehanny-Deierlein damage index for determining impacts of long-duration earthquakes and aftershocks.

**Strength and ductility:**

Strength and ductility play an important role in the structural performance under mainshock-aftershock scenarios. It has generally been observed that a structure's performance deteriorates and collapse capacity decreases as it undergoes more damage during the mainshock. This trend holds for both ductile and non-ductile reinforced concrete frames. However, as found by Raghunandan and Liel [16], modern ductile frames can undergo higher levels of damage before aftershock collapse capacity is seriously compromised. The non-ductile frames are relatively weak and additionally have shear and axial deficiencies along with lack of capacity-protected design. The study by Raghunandan and Liel found that the reduction in collapse capacity as a function of mainshock damage is more precipitous for the non-ductile reinforced concrete frames.

Goda [17] and Goda and Taylor [18] found increases in peak ductility demands of single-degree-of-freedom (SDOF) systems using both real and artificial mainshock-aftershock sequences. Lack of ductility capacity can then lead to worsening of seismic performance or even collapse.

## BRIDGE-SPECIFIC STUDIES:

As clear from the discussion so far, several studies related to mainshock-aftershock performance of buildings have been carried out. It is noted that many of the methodologies and findings would be easily transferable to bridge structures. A few bridge-specific studies have also been carried out to account for the effects of some of the previously described factors on bridge seismic performance. Some of these studies have not utilized the mainshock-aftershock framework but have looked at factors such as earthquake duration, near-field effects, hysteretic behaviour, collapse vulnerability, etc. Such studies are related to bridge performance in the framework of mainshocks but could be easily extended to mainshock-aftershock performance and are summarized below. Other studies have tried to quantify bridge performance in the mainshock-aftershock context. These studies have utilized fragility analysis and bridge functionality loss for assessing risk and resilience of bridges under mainshock-aftershock scenarios.

**Effects of earthquake duration**

It has been observed that bridge columns subjected to long duration motions tend to have a significantly reduced seismic displacement capacity. Sanders et al. [19] and Mohammed [20] carried out experimental and numerical studies to quantify the effect of earthquake duration on reinforced concrete bridge columns. The study found the ground motion duration to have a significant effect on displacement collapse capacity of RC bridge columns. Columns subjected to long-duration motions were on average found to have lower displacement capacities by 25% than same columns subjected to short-duration motions. Spectral accelerations at collapse were found to be significantly affected by the duration of the ground motion with long duration motions having lower spectral accelerations at collapse compared to short ones when applied to the same column.

Ou et al [21] also examined seismic behavior under long duration ground motion in flexural dominated reinforced concrete bridge columns designed in accordance with modern seismic design codes. Two column specimens with identical design parameters were tested using a long duration loading protocol and a baseline loading protocol. The long duration protocol was developed to represent the number of response cycles expected under long duration ground motions. The short duration protocol consisted of one cycle for each drift loading to obtain baseline behavior. Test results showed that the column subjected to the long duration loading had similar peak strength, but a much lower ductility capacity compared to the baseline loading column. The study found that the columns showed a similar hysteretic envelop response up to a of drift 3%. However, the long duration loading column showed a greater stiffness degradation.

Using IDA, Billah et al. [22] investigated the effects of long duration and near fault motions on bridge collapse capacity. The study used a circular reinforced concrete column pier bridge located in Vancouver, BC. The bridge period was determined to be 0.48 seconds. This study found a lower collapse margin for long duration motions compared to near-fault motions. This is due to cyclic strength reduction under longer duration accelerograms. It is noted that this result should be carefully interpreted and not be regarded as a general conclusion; other bridges with different periods subjected to time histories with varying frequency content could lead to a different conclusion.

It is noted that these findings are valuable in terms of drawing basic conclusions for structural response or devising further studies under mainshock-aftershock scenarios. It is recognized that the mainshock and aftershock generally have different frequency content. However, there may be cases of aftershocks with high energy in frequency bands that could match the natural period of the structure damaged during the mainshock. This could then lead to deteriorated seismic performance under the mainshock-aftershock sequence due to the structure being subjected to a longer overall ground motion duration. Further studies would be needed to shed more light on this issue.

**Effects of hysteretic behaviour**

It is imperative to incorporate appropriate hysteretic behaviour in the seismic analysis of structures. Several hysteretic rules are available in literature (e.g. see Priestley et al [23]) and should be used with caution to adequately model cyclic behaviour of elements.

The study by Ou et al [21] found that when drift exceeded 3%, the long duration loading column started to show greater strength degradation. Experimental observations of hysteretic behavior in the two columns found the strength degradation to be related to maximum displacement and energy dissipation. Stiffness degradation was found to be related to energy dissipation whereas pinching behaviour was related to maximum displacement. The authors used the experimental results to construct a new hysteretic model. The proposed hysteretic model was used to carry out constant-R (force-reduction factor) and constant-ductility analyses. The constant-R analysis results showed that ductility demand was not necessarily affected by the long duration ground motions. Ductility demand obtained by the proposed model were generally found to be larger than those obtained by the Modified Clough (MC) model. The difference between the two models increased with the duration of ground motion and R-factor increase and as the period decreased. Constant-ductility plots showed the differences between the two models to be up to 10% and 20% for the short period and long period motions, respectively.

Ruiz-Garcia et al. [24] carried out an analytical investigation aimed at evaluating the inelastic response of typical Mexican reinforced concrete highway bridges (with substructure heights up to 15 m) under main shock-aftershock seismic sequences. The study employed 28 mainshock-aftershock seismic sequences from the Mexican Pacific coast subduction zone. 9 typical configurations of low-height highway bridges in Mexico were modeled and analyzed for the mainshock-aftershock sequences. The study found that the Mexican highway bridges did not experience significant lateral peak and residual drift demands under as-recorded mainshocks. This was attributed to the high inherent structural overstrength and relatively small low-to-medium earthquake intensity. However, the lateral drift demands increased in some cases when aftershocks were considered. Similarly, when scaling seismic sequences to represent stronger ground motion intensities, the level of peak and residual drift demands tended to increase as a consequence of aftershocks. Column strength was found to play a role in case of scaled seismic sequences. The level of increment of peak and residual drift demands depends on the type of hysteretic behavior considered in the columns and the level of ground motion intensity. Different Takeda models such as the Takeda model with no degradation and Takeda models with moderate and severe degradation were employed for this study and showed different results.

A careful selection of hysteresis rule that can capture strength and stiffness degradation and ultimate capacity is therefore critical for mainshock-aftershock performance assessment.

**Seismic fragility analysis of bridges**

Fragility curves can be developed for bridges with and without consideration of the aftershock effects. A fragility curve represents the probability of exceedance of a damage state given a certain seismic intensity measure.

Pang and Wu [25] investigated the effect of aftershocks on seismic responses of multi-span reinforced concrete bridges using fragility analysis. A continuous girder reinforced concrete substructure bridge class containing 8 bridges was selected based on the statistical analysis of the existing reinforced concrete bridges in China. The bridges selected for this study were designed according to the old Chinese bridge design specifications and include limited longitudinal steel with widely spaced transverse ties. All chosen bridges were three span structures with two-column bents having PTEE sliding pads. 75 recorded mainshock-aftershock seismic sequences from 10 well-known earthquakes were selected in this study. The component fragility curves are developed using nonlinear time-history analysis in terms of the peak curvature of pier column and displacement of bearings. The system fragility curves were derived by implementing Monte Carlo simulation on multinormal distribution of the two components. The results from component and system fragilities captured the influence of aftershocks for the studied bridges corresponding to four different damage states, i.e. slight, moderate, extensive and complete. It was found that the influence of aftershock on bridge system changed with the considered damage state. The aftershocks slightly increased the system vulnerability at the slight damage state, while system vulnerability at the moderate and extensive damage states was influenced more extensively by aftershocks. The system fragility was always found to be worse than the component fragility and the study therefore recommends evaluating the bridge system vulnerability rather than the sole assessment of component vulnerabilities due to aftershocks.

Fakharifar et al. [26] and Omranian et al. [27] have used the fragility analysis to assess the vulnerability of retrofitted bridges under mainshock-aftershock scenarios.

Fakharifar et al. investigated the collapse vulnerability of substandard reinforced concrete bridge piers retrofitted with repair jackets comprising fibre-reinforced polymer (FRP), conventional thick steel and hybrid. The hybrid jacket comprises a thin cold-formed steel sheet placed around the column along with prestressing strands wrapped around the jacket thus providing passive and active confinement. This study employed extensive IDAs on a two-span prototype single column bridge bent with lap-splices at the column-footing interface (plastic hinge) zone. Deteriorating properties under repeated mainshock-aftershock sequences were used to determine the effectiveness of different repair jackets. As expected, this study revealed that the original bridge could not satisfy the performance requirements of different seismic codes. The study used both near-fault and far-fault mainshock-aftershock ensembles. It was found that for the severe mainshock-severe aftershock case, the mean structural collapse capacity for the mainshock only versus the mainshock-aftershock scenario reduced significantly showing a 50%-60% capacity reduction in terms of spectral acceleration. This finding is inline with other studies, which have shown the effect of existing damage under a mainshock on the post mainshock collapse capacity. Less reduction in mean collapse capacity was found for the severe mainshock-moderate aftershock scenarios. The jacket retrofits were found to be quite effective for the post-mainshock collapse capacity of the mainshock damaged bridge with an increase in mean structural collapse capacity of 30%-45%. The thick steel jacket was found to be the least effective retrofit measure exhibiting the highest fragility.

Omranian et al. developed fragility curves for the original and FRP retrofitted reinforced concrete bridges under mainshock-aftershock sequences considering only the bridge pier. The skewed Painter Street Overpass in California was used for this study. Fragility curves were developed for four damage states under both mainshock and mainshock-aftershock sequences. Similar to other studies, this study showed a worsening bridge performance when aftershocks are accounted for. The difference between fragility curves under mainshock only and mainshock-aftershock sequences for each damage state showed a more significant impact of aftershocks for cases where the mainshock caused greater damage to the structure. Comparing the fragility of the bridge with and without FRP revealed the positive impact of FRP confinement by improving the bridge seismic performance. The impact was particularly pronounced for the higher levels of damage states such as severe and complete.

**Resilience assessment of bridges**

The drop in bridge functionality can have a significant impact on highway and rapid transit infrastructure systems.

Dong and Frangopol [28] carried out a study based on event identification and seismic vulnerability of the structure tied to its performance. They carried out a risk assessment by determining the social consequences, repair loss and functionality of the bridge as a consequence of the damage. Both mainshock and mainshock-aftershock scenarios were considered. The presented approach helps in the decision making process for: (i) whether or not to open the bridge after an extreme event and (ii) pre-event decision making accounting for proper retrofit strategies for meeting the target resilience and /or risk performance level. It is noted that resilience as a performance indicator attempts to quantify recovery patterns of engineering systems under hazard effects. A two-span reinforced concrete substructure bridge in California was used to illustrate the approach. The study concluded that aftershocks have a significant effect on repair loss and residual functionality of a bridge. The bridge functionality assessment and bridge performance level probabilities can help inform the decision to open bridges after seismic events.

A different yet simple way of quantifying bridge resilience is to determine the reduction in capacity of the structure. There is ambiguity in the definition of capacity and different force or displacement based parameters could be used to determine capacity reduction. Espinosa [29] used the ultimate force, stiffness and strain energy ratios to quantify capacity reduction. This study used a probabilistic seismic demand model (PSDM) framework to assess and compare the capacity reduction of a bridge for both mainshock and mainshock-aftershock scenarios. Pushover analyses for undamaged and damaged states corresponding to mainshock and mainshock-aftershock scenarios were produced. The damaged states were determined using non-linear time history analysis for the mainshock and mainshock-aftershock cases. 3-D surface plots comparing the reduction in capacity ratios conditioned on the peak ground velocity were produced for the mainshock and mainshock-aftershock cases. The dispersion values obtained from the plots are a measure of the appropriateness of the choice of the engineering demand parameter used to predict capacity reduction. The study found the strain energy ration to have the best correlation while the ultimate force was the least well-behaved parameter. Using any of the three EDPs, it was found that the aftershocks cause capacity reduction in the bridge and play a significant role when compared to the mainshock-only scenario.

## CONCLUSIONS

Although the past structural performance under large earthquakes followed by aftershocks has been poor, most seismic codes simply ignore the effects of aftershocks. The design is usually based on a peak engineering demand parameter such as spectral acceleration or displacement. Unlike other codes, S6-14 has incorporated simple but somewhat ambiguous requirements related to seismic capacity retention after the mainshock along with provisions to have full capacity restored by repairs. The quantification of seismic performance under mainshock-aftershock scenarios is a complicated subject as structural and site periods, earthquake duration, near field effects, seismic energy and number of cycles, available ductility and hysteretic behaviour etc. can have vastly different effects on a structure. In addition, non-uniform stiffness and strength attributes are also critical as these are often encountered in bridges and lead to inelastic demand concentrations in certain locations. Several studies have looked at different seismic variables such as ground motion period, seismic sources, near-fault effects, the nature and relationship of real mainshock-aftershock time history combinations, the effects of earthquake direction and polarity, peak structural demands as a function of energy, etc. Structurally, research has focused on the importance of strength and ductility and the associated hysteretic behaviour. Almost all studies have confirmed the worsening of structural performance when aftershocks are considered. This applies to both unretrofitted and retrofitted bridges. The various studies have provided a great wealth of knowledge about the different parameters and how they affect bridge performance under mainshock-aftershock scenarios. However, there is need to unify the approaches and carry out comprehensive studies accounting for effects of different variables simultaneously and in a holistic manner. The authors note that most studies have used extensive non-linear time history analysis to produce trends and formulate conclusions. This rigorous treatment is necessary to develop an in-depth understanding of the issues and impacts on seismic performance of bridges when accounting for aftershock effects and vulnerabilities. Any future studies need to follow a similar course but provide simplified analysis and design recommendations and methodologies for bridge design based on their findings. An example would be to provide procedures and design guidelines that can be used while employing the standard analysis tools required by S6-14. This could include inelastic static pushover analysis for Major-route bridges and limited non-linear time history analysis (using the stipulated 11 time history record approach) for Lifeline bridges. Without such recommendations, the value of further research will be diluted, whereas simplified design tools and procedures will result in the advancement of the state-of-practice leading to the construction of safer and more resilient bridges.

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