SEATTLE DEPARTMENT OF TRANSPORTATION

Bridge Seismic Retrofit Program – Phase III

Bridge Seismic Retrofit Philosophy, Policies, and Criteria

Seattle Department of Transportation

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INTRODUCTION

Earthquake events, occurring over the last several years around the world, show us the magnitude of damage and loss of life that can occur from catastrophic failure of both private and public structures. Additionally, millions of dollars in infrastructure repair costs can be incurred, even for those structures that do not totally fail but that sustain either minor or major damage.

The City of Seattle's roads and bridges form an intricate transportation network that connects the City, which is divided by hills and waterways. Preservation of this network to protect public safety, infrastructure investment, and vitality of the local and regional economy from catastrophic failure or major damage after a seismic event is an important function of the City, which owns and maintains this network via its Seattle Department of Transportation (SDOT). The City's inventory of bridges is highly varied in terms of bridge type, size, complexity, date constructed, materials of construction, and current condition. See Appendix A for the City of Seattle Bridge Inventory Map.

In the early 1990's, inspired by the devastating 1989 Loma Prieta earthquake in the San Francisco Bay Area, the Seattle City Council approved funding and directed the Seattle Engineering Department to analyze the City's bridges, prioritize these bridges and develop retrofit concepts and recommendations. This resulted in the initiation of the SDOT Bridge Seismic Retrofit Program (BSRP).

Phase I of the Program was implemented from early 1990's. Out of 118 bridges, 44 of the most critical bridges were identified for retrofit, and ultimately 23 of these bridges were upgraded to be more resistant to the design earthquake event at the time. See Appendix A for the City of Seattle Bridge Inventory Map. Constrained by budgets, some of the scopes of retrofit were scaled back.

In November 2006, the voters of Seattle passed a levy for transportation maintenance and improvements. This levy, combined with two other sources of funding, made up the Bridging the Gap (BTG) program that dramatically increased available funds for transportation capital projects and needed infrastructure maintenance. The Bridge Seismic Retrofit Program Phase II was part of the BTG program. Seven bridges were selected to be in that phase of the program. These bridges were selected based on their relative traffic importance and structural vulnerability to an earthquake event, using an approach similar to that used in Phase I. In 2008 new seismic peak acceleration maps and pushover analysis were introduced and are now part of the SDOT bridge retrofit procedures.

The purpose of this document is to state SDOT's philosophy and policies as well as establish design criteria for the Program. This document is intended to serve as a link between previous decisions and the current implementation approach, and it provides guidelines for future phases of the Program. Accordingly, this should be a "living" document that will be revised from time to time as more information is obtained.

In 2015, as Phase II neared completion in delivering seismically upgraded bridges, the BSRP Phase III (Move Seattle[1]) was convened and preparing for its launching upon funding availability. The original content of this document is for the large part still valid and kept intact for Phase II and Phase III implementation. This is a "living" document and will continue to be revised from time to time on an as-needed basis.

[1]The Move Seattle Levy was passed in late 2015.

1.0 PHILOSOPHY

1.1 Statement of Philosophy

The philosophy of the SDOT Bridge Seismic Retrofit Program is comprised of the following two guiding principles:

1) First and foremost, provide for public safety in the event of an earthquake and
2) Reduce the socio-economic impact of an earthquake to the community to the extent reasonable.

This statement is intended to apply to the SDOT inventory in general and not solely to the current phase of the program. Therefore, the first principle of the philosophy requires that the entire inventory of bridges be considered and improvements made to correct known deficiencies. It is implicit that each phase of the program represents only a part of the total effort. The second principle provides general guidance, particularly for prioritization. The target level of reduction will be set as high as reasonable. Although the impacts to the community from a large earthquake can be reduced, it should be recognized that they can never be completely eliminated due to limited resources available to the overall program.

In keeping with the long-standing philosophy for new bridge design (Reference 1; Note all references are contained in Appendix B), the general performance objectives listed below should guide the retrofit program:

- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Ideally, damage that does occur should be readily detectable and accessible for inspection and repair, although this may not be achievable for bridges retrofit to Life-Safety performance levels.

- Small to moderate earthquakes should be resisted within the essentially elastic range (defined in Article 8.9 of Reference 5), without significant damage, to the extent reasonable as determined by SDOT. Thus bridges should be usable for emergency vehicles immediately following a smaller event and with limited repairs, usable by the public soon thereafter.

Note that preventing collapse ensures that public life safety is protected, and restricting lower level earthquake damage to repairable locations helps minimize economic impact in terms of repair costs and potential time out of service. The first goal, preservation of life safety, is the most important. The second goal, of immediate use of bridges after a small earthquake, is of lesser importance and should not override the first goal of the retrofit program. This reflects the City's desire to address safety for as many bridges as possible first.

Because earthquakes of unknown intensity may strike at any time, use of retrofit design principles that result in ductile response is highly desirable. This makes the bridge performance less dependent on a precise calculation of earthquake demand. To the extent possible with these existing bridges, retrofit should provide for ductile response of the bridge and should prevent unwanted brittle failure modes from occurring, to comply with capacity design principles.

1.2 Goal for Inventory

From a completeness standpoint, it is important that all of the City's bridges be considered for seismic retrofit. The minimum seismic safety goal for the City's inventory is that every bridge should meet the life-safety performance objective of no-collapse in the design earthquake, where the "design earthquake" was an event with a 500-year return period for Phase I and is an event with a 1000-year return period for the Phase II & III programs, following accepted seismic retrofit guidelines at the time of Phase I, Phase II and Phase III implementation. The 1000-year return period seismic event is the design earthquake event for Phase III.

1.3 Communications

The current retrofit program will only address a portion of the total bridge inventory in terms of retrofits that are constructed, so an effort should be made to preserve as much of the thinking, decisions, and results as possible to aid in continuity with future retrofit efforts. For example, this document will be kept and revised by SDOT as new information is obtained and agreed upon. Additionally, Design Reports will be written for each bridge that is retrofitted. These reports will contain information and details for the retrofits, including the potential for damage in the lower level earthquake. Guidance regarding potential locations and types of damage will also be provided in the reports to aid inspections of the retrofitted bridges after future earthquakes. All contract documents, including drawings, specifications, calculations and inspector's reports will be maintained for each retrofit project by SDOT. A Post-Earthquake inspection report shall be generated for each retrofitted structure.

2.0 SDOT SEISMIC RETROFIT POLICY

General: SDOT has established the following seismic retrofit policies that provide over-arching principles that will guide the SDOT Bridge Seismic Retrofit Program in its implementation. These policies are indicative of issues where SDOT will provide overall decisions and guidance to the project teams. These may be changed by SDOT on a case-by-case basis.

> Commentary: It is expected that the project teams will work closely with SDOT to obtain appropriate and timely decisions regarding retrofit objectives and approaches. The seismic retrofit performance objectives have been purposely selected to be somewhat subjective for this program. Current guidelines for seismic retrofit assessment and design may include multiple levels of design earthquakes to be considered along with varying performance objectives based on bridge importance and anticipated service life. These earthquake levels and the associated performance are quite specific and may not be readily or evenly achievable for some older structures. Thus maximum flexibility should be afforded the designers of potential retrofits such that they can identify and potentially implement the most cost-effective and structurally effective measures. Each structure must be considered individually, with recognition of unique features, condition, and detailing. Structures vary significantly, and written guidelines, particularly prescriptive ones, will not be effective in all cases in terms of identifying the most vulnerable parts of a structure. This must jointly be SDOT's and the designer's responsibility – the designer's to frame alternatives and costs, and SDOT's to decide accordingly.

2.1 SDOT Seismic Retrofit Policies

2.1.1 Bridge Importance

Policy: All City bridges will be considered ***Standard***. Classification of ***Essential*** will be on case-by-case basis by SDOT.

> Commentary: A common distinction made for bridges is to separate so-called Essential or Important structures from the rest and assign higher performance objectives to these structures. Conditions that potentially cause a bridge to be considered Essential are:
>
> - The bridge is required to provide emergency access to services such as hospitals.
> - Loss of functionality of the bridge would create a major economic impact to the community.
> - The bridge is formally defined as part of a local emergency plan and no alternate routes exist.
>
> Typically, to meet the higher standards for Essential bridges significantly enhanced features, and thus cost, will be required. Because of this, designation as Essential should be undertaken only with the utmost care to avoid using excess resources.

2.1.2 Earthquake Hazard Levels and Performance

Policy: Two earthquake levels will be considered, one with a 100-yr return period (Lower Level Earthquake) and one with a 1000-yr return period (Upper Level Earthquake). The Upper Level earthquake is to be used to design seismic retrofits for the "Standard" classification of bridges, eg. 1000-yr return period for life-safety protection. Retrofitting measures for the Lower Level earthquake may also be considered to maintain essentially operational performance – essentially elastic response - subsequent to the smaller event. The Lower Level earthquake shall be assessed for each bridge, but retrofit design to meet the stated performance goal will only be undertaken at the direction of SDOT.

Those bridges classified as “Essential” will be designed to remain essentially operational subsequent to the Upper Level earthquake. In the context used herein, ‘essentially operational’ means that some limited disruption of service is permitted.

Commentary: Note that for many years the 500-year earthquake (15% chance of being exceeded in 75 years) has been the design event. The proposed goal of the program is to recognize the work previously completed using that design event. However in 2007, AASHTO (Reference 4) adopted a design ground motion corresponding to a return period of 1000 years for new design. The 1000-year return period corresponds to approximately a 7% chance of exceedance in 75 years. The actual return period used by USGS to develop 1000-yr maps is 975 years. See Appendix C for further information regarding the design earthquake. Additionally, the most recent FHWA guidelines for seismic retrofit (Reference 3) have also adopted the 1000-yr event as its basis for its recommended seismic hazard. In transitioning to the longer return period, the design procedures did not remain the same. To compensate for the larger demands, some of the conservatism in capacity was removed; thus one should not infer that the costs of new design or retrofit are directly proportional to the increase in seismic hazard level.

The current seismic retrofit guidelines published by FHWA and MCEER *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges* (Reference 3) are based on a two-level approach, whereby for standard bridges operational performance is desired for a smaller, more frequent earthquake and life-safety/no collapse is desired in a larger event. From a policy standpoint, the achievement of life-safety response should control over operational response in smaller earthquakes. While it is recognized that loss of use of a structure in a smaller earthquake has a significant effect on the local economy, the prevention of loss of life is the primary concern. This observation is reflected in the new LRFD provisions for seismic design, including the Guide Specification for new design (Reference 5), whereby a single level earthquake is used for design of new structures; no service level earthquake is mandatory for consideration. Thus, a lower service level of earthquake may be evaluated, but based on costs, retrofitting for a lower level earthquake may be considered optional. The reason for this is to improve the greatest portion of the inventory with the monies available.

Several versions of national hazard maps are available from USGS, as they have continually up-dated their hazard mapping over the years. The *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges* reference the 1996 maps, and the recently adopted AASHTO Guide Specification (Reference 5) and AASHTO LRFD 2008 Interim (Reference 4) use the 2002 maps. A CD tool that is part of the AASHTO website can be used to directly determine the 1000-year ground motions. The USGS/AASHTO 1000-year Seismic Hazard Maps included in References 4 and 5 are dated 2007. They are the same as those on the CD tool, and they are based on the 2002 USGS Seismic Hazard maps. These data are based on the 2002 USGS maps. The 100-year design spectra can be determined from the USGS Seismic Hazard website using either the Seismic Hazard website tool or the Interactive Deaggregation tool. See Appendix G for a detailed discussion of the methodology for this determination.

In 2008, the USGS issued updated hazard maps that include the new NGA (Next Generation Attenuation) relationships along with the hazard from additional faults added to their database. While these new maps are not as convenient to use to develop the two ground motions used for this program, it is permissible to use the NGA maps, but only with SDOT’s concurrence.

2.1.3 Anticipated Service Life

Policy: A bridge that has 15 years or less of anticipated service life or is scheduled for replacement within the next 15 years will be considered by SDOT on a case-by-case basis for retrofit.

For bridges with more than 15 years of anticipated service life, as determined by SDOT, the seismic design hazard levels shall be the basic Upper and Lower Level earthquake ground motions.

Commentary: It is recognized that due to the varied nature and condition of the SDOT bridge inventory not all bridges will have the same anticipated service life (ASL). The current FHWA guidelines suggest a 1000-year return period for all bridges with an ASL over 15 years. If the ASL of a bridge is determined by SDOT to be less than 50 years and retrofit costs for upgrade to a 1000-year return period are prohibitive, it might be appropriate to consider an 'equal hazard' approach whereby the 7 percent chance of exceedance would be used in conjunction with the ASL of the bridge. Due to the inherent difficulty in establishing ASL, a stepped approach could be used. For instance, three ranges might be considered 15-30 years, 30-50 years, and 50 or more years. However, due to the difficulty in establishing appropriate ASL and the relative importance of the bridge to the larger infrastructure system, such an approach should only be used as a last resort, on a case-by-case basis, and only as directed by SDOT.

In general, bridges with less than 15 years or so of life are not given a high priority for retrofit, and the FHWA guidelines do not recommend retrofit consideration for bridges with a 15-year remaining life or less. However, some SDOT bridges that have limited remaining life also may have significant importance to the community. Therefore, each bridge will be considered by SDOT based on its own vulnerabilities, importance and likelihood of future replacement. Additionally, if inexpensive retrofit measures, for example restrainers, provide significant enhancement to life safety, then such retrofits may be considered. In the past, bridges with limited life have been retrofitted to provide life safety, even though the remaining life of the structure is relatively short. SDOT does not have full control over the future replacement timelines; therefore interim retrofits are sometimes warranted.

2.1.4 Age of Bridge

Policy: Bridges built after 1983 and which were designed with the AASHTO 1983 Seismic Guidelines for Highway Bridges or later versions will not be considered as high priorities for retrofit over those designed prior to 1983.

Commentary: Beginning in the mid-1980's new bridges were designed using the newly developed guidelines of ATC-6 (Reference 2) subsequently adopted as a Seismic Guideline by AASHTO in 1983. Thus the logical breakpoint for consideration of potential retrofit of bridges for the City is this point in time where the ATC-6 provisions were first applied to the design of new bridges in Seattle. The ATC-6 design guidelines were subsequently adopted into the AASHTO Standard Specifications in 1990 (Reference 1) following the 1989 Loma Prieta earthquake in California.

Bridges designed and constructed after the implementation of the ATC-6-based design provisions should be considered 'compliant' with modern seismic design codes. Conversely, older bridges whose design pre-dated the use of the ATC-6-based approach should be considered non-compliant and thus should be evaluated or considered for seismic assessment and potential retrofit.

Even though bridges designed after the 1971 San Fernando, CA earthquake may have benefitted from interim improvements to the seismic design specifications, it is often difficult to discern what was actually incorporated into the final design. Thus unless clear information otherwise exists on the drawings or in available calculations, such bridges should be considered non-compliant.

The term 'compliant' above is used to convey the idea that ordinary bridges designed using the various specifications that have followed ATC-6 (i.e. 1983 AASHTO Seismic Guidelines, AASHTO Standard Specification, Division I-A, and the AASHTO LRFD seismic specifications) should essentially have similar detailing and design forces. This statement is rooted in the fact that all these design specifications refined the initial seismic design procedures over the years. The practice and implementation of these specifications, however, have evolved somewhat since the early 1980s. Therefore, bridges designed in the past few years should have better overall detailing and seismic design features than those from the 1980's.

Some aspects of bridges constructed after 1983 may still be vulnerable to earthquake loading. Such aspects reflect areas where design practice has advanced since the early 1980's. Examples include the procedures used to consider liquefaction effects and the understanding of near-fault and vertical acceleration effects. Therefore, a panel of experts will be convened to screen bridges built after 1983 for potential vulnerabilities. The bridges previously retrofitted should also be included in this screening.

2.1.5 Bridge Location / Usage

Policy: Small neighborhood bridges or pedestrian bridges over minor, non-lifeline roadway, as determined by SDOT, will not be retrofitted until all other bridges are fully seismically upgraded.

Commentary: Small neighborhood bridges and pedestrian bridges over minor roads or creeks, whose consequence from damage during a design level earthquake is considered small, are a low priority for seismic retrofitting. However, larger bridges that carry or cross over arterials and larger neighborhood bridges will be considered for seismic retrofitting by SDOT at some future time.

2.1.6 SDOT Owned Bridges

Policy: SDOT will only consider bridges owned by SDOT for seismic retrofitting.

Commentary: Bridges that are owned by other City of Seattle Departments and are only inspected by SDOT are not considered the responsibility of SDOT for seismic retrofitting.

2.1.7 Holistic Retrofit Approach

Policy: The City will not adopt the phased approach to retrofitting, whereby several different construction efforts are required to implement the retrofits, but rather will develop a complete list of retrofit needs for a given bridge and implement them at one time.

Commentary: Some agencies have utilized a phased approach for improving the seismic safety of their inventories. For instance, both California and Washington State DOTs have installed superstructure retrofits first to prevent unseating, then at a later date installed substructure improvements, such as column jacketing, followed at a still later date potentially with foundation improvements. Such an approach permits funding to be spread somewhat evenly over the inventory. In Phase I, SDOT used an approach where retrofits were completed on a given bridge once work was begun. This approach was continued in Phase II and will be continued in Phase III. The phased approach will not be applied to SDOT's inventory, which is highly non-uniform in type, size and material of construction.

2.1.8 Performance-Based Retrofit

Policy: The City will adopt a performance-based seismic retrofit approach for its bridges. It is desirable to capacity protect Live Load carrying elements whenever possible (keep the ductility and strains in the columns). To the extent possible, more advanced displacement-based assessment techniques, such as pushover (nonlinear static analysis) shall be used as a minimum. Where appropriate and where available, performance-based strain or deformation limits shall be used. Additionally where reasonable, improvements such as base isolation should be considered.

Commentary: Several approaches for seismic resistance improvement could be considered, and these generally will include improving displacement capacity, limiting forces that can be generated, or modifying the response to fundamentally change the dynamics of the structure.

Material strain or deformation limits shall be used where they are appropriate based on the type of construction being considered. It is recognized that such limits may not be available for all structure types likely to be found in the SDOT inventory. Limits for construction outside of that for which the limits were developed should be agreed to between SDOT and the project team. In some cases, testing may be required to develop appropriate limits, and such testing should be undertaken only with SDOT direction and concurrence.

2.1.9 Seismic Retrofits for Bridge Preservation

Policy: Seismic retrofit is viewed as a bridge preservation strategy by SDOT, and not a general transportation improvement strategy; thus, under the BSRP only the bridge features that require retrofitting to satisfy seismic performance objectives will be constructed.

Commentary: The Bridging-The-Gap Seismic Retrofit Program (Phase II) fund was solely a resource for SDOT bridge maintenance and was applied to mitigate seismic risks, only. As of July 2008, no other funding was available to address other improvement needs of the bridges in Program. This situation is the same as that which prevailed during the Phase I BSRP and is expected to be the same in Phase III (Move Seattle). In general, funding allocated to the Seismic Retrofit Program will be preserved to serve its use.

2.1.10 Bridges Previously Retrofitted

Policy: Bridges that were retrofitted in previous phases to either operational or life safety levels, and for which all identified retrofits were constructed, will not be retrofitted further.

Commentary: Because retrofit work was previously constructed, but not necessarily completed, continuity of new retrofit work with previously completed retrofits should be considered to the extent possible. Thus, work previously completed will be leveraged to produce a greater value to the community than if the remaining work was left undone.

2.1.11 Subsurface Foundation Retrofit

Policy: Investing in subsurface retrofit will be a low priority unless the life-safety performance level cannot be met without such retrofit. Life-safety performance of the foundation (i.e. the ability of the foundation to provide gravity load carrying capacity after seismic damage is incurred) shall be evaluated.

Commentary: To effectively leverage monies for as many structures as possible, foundation retrofits should only be considered if there is a clear potential for a life safety threat in the event of foundation failure. In many cases failures in the foundations may not lead to life safety hazards because bridge systems are often redundant enough to redistribute the forces that a failed foundation element would otherwise carry. The potential for differential settlement that could cause distress in continuous superstructures should be considered as part of the life-safety evaluation. Because redundancy, continuity and detailing play such a key role in establishing whether foundations should be considered for retrofit, each structure should be evaluated separately.

2.1.12 Retrofit vs. Replacement Cost Considerations

Policy: If cost of retrofit is a substantial fraction of the replacement cost for a given bridge, SDOT may choose not to retrofit and instead program the bridge for replacement.

2.1.13 Movable Bridges

Policy: Movable bridges will be evaluated for both the Lower Level and Upper Level Earthquakes in the closed position. The general performance objectives are to retain functionality, both structurally and mechanically, in the Lower Level event and to preserve Life Safety in the Upper Level event. Following an initial evaluation of a bridge for these two events, the deficiencies and proposed retrofit concepts will be identified. SDOT and the project team will then develop a strategy for reduction of seismic risks posed by the bridge.

Commentary: The City's inventory includes several movable bridges. Most of these movable bridges were constructed early in the 1900's well before any seismic design requirements for bridges were available. It is therefore realistic to expect that achieving operational performance in the Lower Level event may be challenging and that to expect operational performance in the Upper Level event may be unattainable without expending significant resources. Furthermore, each movable bridge is unique. Therefore, it is expected that the performance objective and retrofit strategy may be different for each bridge. The project team and SDOT will be required to jointly develop a strategy that provides the best value and best risk reduction for the resources available. At a minimum, preservation of Life Safety in the Upper Level event should be met.

Consideration of the closed position is mandatory, because that is the position in which the movable portion spends most of its time, and that is the position that is of primary importance to life safety. Other positions may be considered, depending on the frequency and duration of the movable portions of the bridge being in those positions, as agreed to by SDOT. Vertical accelerations should be considered for the leaf, counterweight, and trunnion beam when present in order to mitigate any safety concern for these elements. The massive bascule piers should not be retrofit for vertical accelerations without approval by SDOT. Such retrofit may not be cost-effective, but also could have created displacement issues for non-seismic load cases and service conditions. The designers may identify potential deficiencies due to vertical accelerations, evaluate the risk vs. gain, inform the City accordingly, and make recommendation to the City for retrofit.

2.1.14 Liquefaction

Policy: The potential for liquefaction and liquefaction-induced hazards, such as lateral spreading, will be identified on a project-specific basis by the geotechnical designer for the project. The effect of liquefaction on lateral resistance and vertical support of the structure shall be considered. Potential mitigation measures as required to ensure life safety shall be proposed to SDOT and will be evaluated by SDOT on a case-by-case basis.

Commentary: Liquefaction-induced effects in terms of altered ground motion altered lateral resistance and altered axial capacity should be considered because these can have a significant effect on the response and performance of bridges. Where the potential for such effects exist the project team should work with SDOT to develop an appropriate approach, including the development of site-specific ground motion, if warranted.

Where the potential for lateral spreading of flow, or other site soil movements such as slope stability or downdrag exist, mitigation of such effects may be considered. However, such mitigation can be expensive and may involve property beyond the limits of the City's facility. Therefore, consideration of mitigation of and/or structural design for site soil movements should be handled on a case-by-case basis with close coordination between the structural, geotechnical and SDOT engineers.

2.1.15 Live Load Concurrent with Seismic Loading

Policy: Live load shall only be considered concurrently with seismic loading where the presence of live load produces conditions that significantly influence life safety. The project team shall evaluate such a condition and obtain concurrence with SDOT regarding inclusion of live load.

Commentary: Under most conditions live load does not significantly reduce the ability to achieve life-safety performance in the Upper Level earthquake. However, in some instances this may not be the case. An example of particular concern is beams that may not have sufficient capacity to force plastic hinging (damage) into the columns and which then may fail in brittle modes, such as shear. In such cases, at least 30% of the live gravity load should be considered simultaneously with earthquake loading. However, it is not necessary to consider this same load as part of the seismic mass, because the live load usually decouples dynamically from the movement of the primary mass of a bridge.

The definition of “significant” with respect to the live load’s effect on life safety is relative. The evaluating engineer should consider this on a case-by-case basis depending on the structural system, redundancy, load levels and the type of deficiency. Considerable judgment may be required. Therefore, no specific trigger with respect to capacity and demand is provided herein.

2.2 Prioritization

The SDOT bridge inventory has been screened to identify structures that are seismically deficient and to prioritize them in order of importance for seismic retrofitting. This was done in Phase I and Phase II. Entering into Phase III, the inventory will be screened again to be prioritized based on the order of importance and in conjunction with the BSRP’s policies. A prioritization is required to match the funding available with bridges that are ranked with the highest consequence to the community should a seismic event damage those bridges.

The program parameters considered in the prioritization process are identified below. Prioritization in Phases I, II and III of the retrofit program have been accomplished by expert panels knowledgeable in structural engineering, traffic engineering, and emergency response that were convened by the SDOT specifically to prioritize its bridges for seismic retrofit. The items and parameters listed in this section are provided as general guidance for use in the prioritization process. Each item would be considered on its own merit for the particular bridge being considered in the prioritization process.

2.2.1 Traffic Considerations

A relative ranking of the bridge inventory according to their traffic importance should be a consideration for seismic retrofitting. Routes on the bridges as well as those that pass under should consider the following:

- Transit routes – Major transit routes have higher importance if detours are unavailable.
- Freight routes – Major freight routes have higher importance if detours are unavailable.
- Available detours – Bridges with available detours or those detours that could quickly be constructed after a design level earthquake have a lower priority.
- Average daily traffic (ADT) – A relatively high ADT will affect more of the traveling public and have a higher importance than a relatively low ADT.
- Critical emergency routes – Critical emergency routes have higher importance if detours are unavailable.
- Economic impact – Those routes that connect major economic centers within the City are considered to have a higher importance.

2.2.2 Structural Considerations

A relative ranking of the bridge inventory according to each bridge’s seismic vulnerability should be another consideration for seismic retrofitting. The following structural vulnerabilities should be considered:

- Bridge Age
- Structure Type (redundancy, ductility)
- Structure Materials (brittle/ductile, material properties)
- Features likely to fail during a seismic design event (lateral load path discontinuities, insufficient bearings, non-ductile members, non-symmetrical lateral stiffness)
- Previous seismic retrofits constructed

- Structure condition (based upon inspection reports)
- Adjacent Structures (consider impact of adjacent structures on seismic performance)
- Previous earthquake performance (based upon inspection reports)
- Geotechnical hazards (earthquake-induced settlement, liquefaction-induced lateral spreading, and slope stability)

2.2.3 Environmental Considerations

The impact to the environment of a non-retrofitted bridge versus a retrofitted bridge should be another consideration for the seismic retrofitting of projects. The following environmental concerns should be considered:

- Permitting
- Site contamination
- Hazardous waste
- Potential utility contamination
- Biological issues
- Water contamination issues
- Aesthetics
- Historical issues
- Archeological issues

2.2.4 Summary

Based upon the above program parameters, qualified experts in conjunction with SDOT have prepared relative rankings. These rankings, in combination with the Bridge Seismic Retrofit Policies listed above, provide the basis for the seismic retrofitting prioritization of the City of Seattle's bridges in the Bridge Seismic Retrofit Program. For future retrofit efforts, the prioritization for Phase III (Move Seattle) has been revisited by SDOT to determine which bridges will be selected for assessment and potential retrofit.

3.0 INTRODUCTION TO SEISMIC RETROFIT CRITERIA

3.1 Scope

Part III of this document establishes Criteria for the structural engineering design associated with the Seattle Department of Transportation (SDOT) Bridge Seismic Retrofit Program. These Criteria should be used to achieve an economical retrofit solution for the bridges identified in the Program.

These Criteria do not substitute for engineering judgment and sound engineering practice. The designers are responsible to identify any necessary departure from the Criteria contained in this document, and to bring it to the attention of SDOT. Any changes to these Criteria must be reviewed and approved by SDOT prior to use in the retrofit design. Application for changes to this document, additions to this document, and other questions should be submitted in writing to SDOT.

3.2 Reference Documents

The bridges undergoing seismic assessment and potential retrofit construction, which are governed by these Criteria, are within the City Limits of the City of Seattle. It is the express intent for these Criteria to be in substantial compliance with the codes and standards of the local jurisdiction. With the exceptions of SDOT policy, as listed in Part II of this document, or specific exceptions provided in this Part III of the document, all retrofitted bridge structures shall be designed in accordance with the codes, standards and specifications listed below:

1. Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges (Publication No. FHWA-HRT-06-032, (January, 2006))

2. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (2011), with 2012, 2014 and 2015 Interim Revisions

3. AASHTO LRFD Movable Highway Bridge Design Specifications, 2nd Edition (2007), with 2008, 2010, 2012, 2014, and 2015 Interim Revisions

4. WSDOT Bridge Design Manual LRFD, (M 23-50.14), April 2015

5. WSDOT Geotechnical Design Manual, (M 46-03.07), April, 2014

6. AASHTO LRFD Bridge Design Specifications, 7th Edition (2014), with 2015 Interim Revisions

The order of precedence of the design specifications shall be as shown above. The editions listed above are the current publications available at the time of this report. The latest updates to these documents should be used or the specific version of the document being used by the current SDOT bridge program phase.

4.0 BASIS OF DESIGN

Each bridge seismic retrofit shall have a basis of design document, prepared using the *Part III – Bridge Seismic Retrofit Criteria*. Included in this document shall be a *Seismic Retrofit Bridge Data Summary*. See Appendix F for guidelines and an example.

4.1 Design Earthquake

The *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges* uses two design earthquakes. Following are the definitions of the two design earthquakes:

<u>Lower Level (LL) Design Earthquake</u> – Lower-level design event that has ground motions corresponding to a 50% probability of exceedance in 75 years or an approximate nominal return period of 100 years.

<u>Upper Level (UL) Design Earthquake</u> – An upper-level event that has ground motions corresponding to a 7% probability of exceedance in 75 years or an approximate nominal return period of 1000 years.

4.2 Performance Level

A Performance Level provides a means of communicating risk and potential damage along with the necessary engineering information to develop retrofit designs consistent with stated performance goals. The service and damage levels are defined for each design earthquake in terms of a Performance Level. Two separate factors determine the Performance Level of a bridge structure: Importance and Anticipated Service Life.

4.3 Bridge Importance

The bridges identified in this Program shall be classified by SDOT in accordance with the *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*.

4.4 Anticipated Service Life

The anticipated service life of a bridge is an important factor when deciding the extent to which a bridge shall be retrofitted. Estimating the anticipated service life of each bridge identified in this Program shall be done in consultation with SDOT.

4.5 Seismic Retrofit Category (SRC)

Performance Level and Seismic Hazard Level are used together to determine the appropriate Seismic Retrofit Category (SRC) for each bridge at each level of design earthquake. The SRC guides the strategy for screening, evaluating, and retrofitting deficient bridges.

The Seismic Retrofit Category for each bridge identified in this Program is to be included in the Seismic Retrofit Bridge Summary document, see Appendix F. These categories have been derived in accordance with the recommendations of the *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*, and should be used in bridge screening, evaluation and retrofit design strategies going forward.

A SRC of less than C shall not be used without SDOT's concurrence.

4.6 Material Properties

Expected material properties as defined in the WSDOT Bridge Design Manual (LRFD) shall be used for analysis only. Retrofit design shall use the specified design values for each bridge material.

- Expected concrete strength ($f'_{ce} \sim 1.5\, f'_c$). A 50% increase in concrete compressive strength should be used to calculate the modulus of elasticity and to evaluate the shear and flexural capacities of the aged concrete provided the condition of the concrete is sound. Otherwise, strength should be assessed directly.

- Ultimate strain limits for compression failure of unconfined concrete shall be 0.005 in accordance with *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges* Section 7.8.2.1. The tensile strain limit for longitudinal reinforcement shall be 0.10 in accordance Section 7.8.2.4.

- Use expected steel strength ($f'_{ye} \sim 1.1\, f_y$). The actual tested steel yield strength is assumed to be higher than the nominal strength by 10%.

Where older types of material have been used, for instance square bars, straight or twisted, deformed or not with unknown material properties and unknown bond characteristics, case-specific data may need to be developed from testing of coupons or from relevant literature.

4.7 Load and Resistance Factors

In accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design and WSDOT Bridge Design Manual (LRFD), load factors of 1.0 shall be used for all permanent loads. The live load factor for Extreme Event – I Limit State load combination, γ_{EQ} as specified in the AASHTO LRFD Table 3.4.1-1 for all bridges, shall be equal to 0. The γ_{EQ} factor applies to the live load force effect obtained from the bridge live load analysis. In certain cases, where live load may produce significant deleterious effects (e.g. outriggers, overhangs, horizontal elements that are shear critical) then a factor of 0.30 should be used (See Section 2.1.15). The associated mass of live load need not be included in the dynamic analysis. All resistance factors, Φ, shall be taken as 1.0 except as noted in the WSDOT Geotechnical Design Manual.

4.8 Numerical Analysis

In general the procedure to analyze the capacity demand ratios (C/D) for the bridges follows the displacement based methodology as outlined in *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges* Section 5.6 Method D2. This methodology is a three-step process. First, a nonlinear static procedure or pushover analysis is made to evaluate the displacement capacity of the individual bridge piers. This analysis step considers each relevant limit state and level of functionality, including P-Δ effects. Second, an elastic response spectrum analysis is made to assess the displacement demands of the bridge. Then the capacity and demand results from steps 1 and 2 are compared to determine C/D ratios at each critical limit state. It is the intent of this assessment to focus on displacement rather than force design. Time history analysis is not envisioned, unless approved by SDOT.

Bridge seismic analysis shall be performed using one of the following structural finite element analysis programs, unless an alternative acceptable program is approved by SDOT:

- SAP2000
- LARSA4D
- GT-STRUDL
- CSiBridge

4.9 Foundation Modeling

Foundation modeling shall be performed in accordance with the *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges,* Section 6.2 Foundation Modeling. Deep foundation (soil spring) analysis shall be performed using one of the following analysis programs, unless an alternative acceptable program is approved by SDOT:

- FB-MultiPier
- LPile
- Group

5.0 GEOTECHNICAL

Geotechnical recommendations at each bridge site will be issued by the geotechnical engineer of record in support of the retrofit design process. These recommendations will include Soil Site Classifications and accompanying vertical and horizontal design response spectra for the two design earthquake levels. The design response spectra are used to determine the Seismic Hazard Level in accordance with the *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*. See Appendix F for the Required Geotechnical Data Parameters to be provided for each bridge.

The geotechnical report will also identify and recommend mitigation measures as required for geotechnical hazards as follows:

- Liquefaction
- Liquefaction induced hazards, such as lateral spreading and loss in lateral support
- Soil Settlement (Including downdrag)
- Surface Fault Rupture
- Slope Stability

Recommended soil parameters in support of foundation soil spring derivation will also be prepared. In areas of potential liquefaction, both the liquefied and non-liquefied soil properties will be provided.

Vertical design spectra will be considered only if it is critical to integrity of the structure, such as single column bridge substructures with large overhangs and bascule bridges. Site Specific Response Spectra should be considered on a case by case basis.

6.0 SPECIAL BRIDGE CONSIDERATIONS

This section is only applicable to special bridge considerations as follows:

6.1 Movable Bridges

6.1.1 Design Earthquakes

The Lower Level and Upper Level Design Earthquakes, as defined in Section 4.1 shall be used to evaluate bascule bridges.

6.1.2 Performance Levels

Unless the bridge classification is Essential or directed otherwise by SDOT, the following criteria should be used.

Lower Level Earthquake: Because the operation of movable bridges is sensitive to distortions and misalignments, the movable span and associated supporting elements should be evaluated and retrofitted to meet Performance Level 3 (PL3) in accordance with the *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*. In this way, the bridge should sustain negligible damage and should remain fully operational following the Lower Level Design Earthquake.

Upper Level Earthquake: The bridge shall be evaluated for the Performance Level 1 (PL1): Life Safety for the Upper Level Earthquake in the closed position. The bridge is not expected to be operational nor even repairable after such and earthquake.

6.1.3 Seismic Retrofit Category (SRC)

Performance Level and Seismic Hazard Level are used together to determine the appropriate Seismic Retrofit Category (SRC) for each bridge and design earthquake. The SRC guides the strategy for screening, evaluating, and retrofitting deficient bridges.

6.1.4 Analysis Configuration

The design level earthquakes shall be considered in the closed position. Other positions of the bascule leaves or movable portion may be considered on a case-by-case basis, depending on the frequency and duration of the leaves or span being in those positions. The criteria for such considerations shall be coordinated with SDOT.

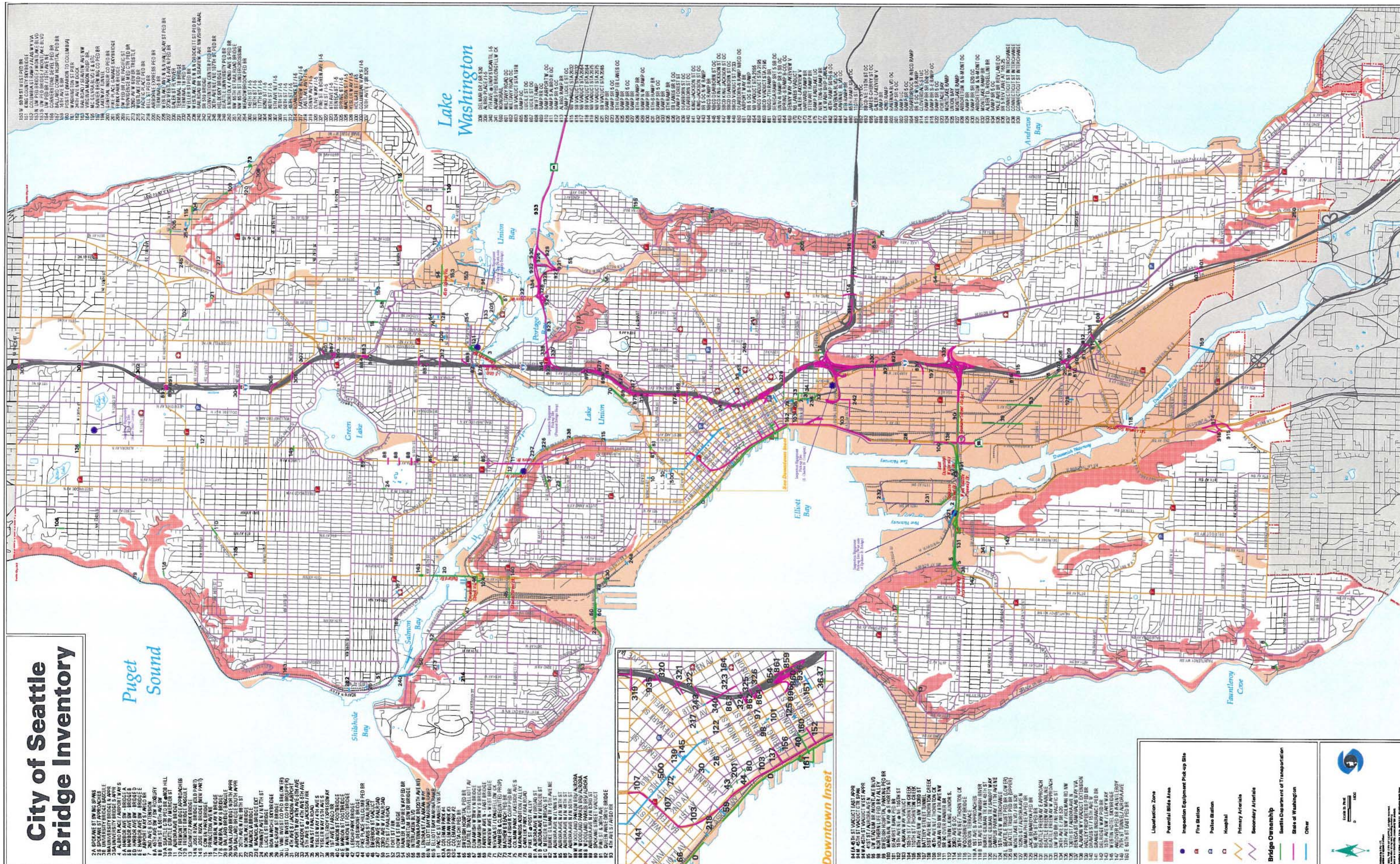
6.2 Seismic Isolation

The use of seismic isolation shall be considered on a case by case basis with the approval of SDOT.

APPENDICES

APPENDIX A

CITY OF SEATTLE BRIDGE INVENTORY MAP


City of Seattle
Bridge Inventory
Lake Washington
Union Bay
Portage Bay
Lake Union
Green Lake
Puget Sound
Salmon Bay
Shilshole Bay
Elliott Bay
Andrews Bay
Fauntleroy Cove
Duwamish Waterway
Downtown Inset
Liquefaction Zone
Potential Slide Area
Inspection Equipment Pick-up Site
Fire Station
Police Station
Hospital
Primary Arterials
Secondary Arterials
Bridge Ownership
Seattle Department of Transportation
State of Washington
Other

APPENDIX B

REFERENCES

APPENDIX B - REFERENCES

1. AASHTO *Standard Specifications for Highway Bridges*, Division I-A, Seismic Design.
2. Applied Technology Council, ATC (1981) ATC-6, *Seismic Design Guidelines for Highway Bridges*, Redwood City, CA.
3. Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges (Publication No. FHWA-HRT-06-032, (January, 2006))
4. AASHTO LRFD Bridge Design Specifications, 7th Edition (2014), with 2015 Interim Revisions.
5. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (2011), with 2012, 2014 and 2015 Interim Revisions
6. Federal Highway Administration, FHWA (1995) *Seismic Retrofit Manual for Highway Bridges*, Publication No. FHWA-RD-94-052.
7. WSDOT (2007) *Bridge Seismic Retrofit Program*, Status Report.
8. Caltrans Seismic Advisory Board (2003) *The Race to Seismic Safety, Protecting California's Transportation System*, Caltrans.
9. Caltrans (2001), Chapter 7, Seismic Safety Retrofit, *Local Assistance Program Guidelines*, Caltrans.
10. AASHTO LRFD Movable Highway Bridge Design Specifications, 2nd Edition (2007), with 2008, 2010, 2012, 2014, and 2015 Interim Revisions
11. AASHTO Guide Specifications for Seismic Isolation Design, 4th Edition (2014).
12. Caltrans – Memo(s) to Designers (MTD)
13. Caltrans – Seismic Design Criteria (SDC)
14. Caltrans – Bridge Design Criteria (BDC)
15. Caltrans - Closing the Gap in the Race to Seismic Safety Report, 2010
16. WSDOT Bridge Design Manual LRFD, (M 23-50.14), April 2015
17. WSDOT Geotechnical Design Manual, (M 46-03.07), April, 2014

APPENDIX C

UNDERSTANDING THE DESIGN EARTHQUAKE OR GROUND MOTION

APPENDIX C - UNDERSTANDING THE DESIGN EARTHQUAKE OR GROUND MOTION

The design earthquake or design ground motion (e.g. a response spectrum as provided by Reference 3) in this document is characterized using a term called return period. The relationship between return period and some exposure period, for example remaining life of a bridge, should be considered. Return period defines the average time between the occurrences of extreme events, such as earthquakes, when considered over a very long period of time on a human (or infrastructure) time scale. The key is the word 'average'. Such events can occur in short succession or be spaced out over very long time periods; nature does not operate with clock-like repetition.

However, return period is simple to use in discussion; although it is somewhat more difficult to comprehend in reality. Thus, an alternate way to describe the occurrence of earthquakes is the chance or odds of an earthquake occurring within a set period of time. The time period can be set to a duration that is meaningful to humans and their infrastructure. The two methods, return period and chance of occurrence, are different ways of characterizing the same thing. For example, an earthquake with a return period of 1000 years has about a 7 percent chance of occurring or being exceeded in 75 years. Thus, if we keep the probability of exceedance at a constant 7 percent, then for a 30-year time window, the corresponding return period would be about 400 years. Note the change in terms from occurrence to exceedance – meaning we also account for the chances of bigger earthquakes occurring.

A given effect of an earthquake occurring at a site, ground acceleration for example, over a given time frame is known as the seismic hazard. Thus, we do not design for a specific earthquake. Instead we design for a given chance that some damaging level of movement occurs in a set time frame. For example, the design earthquake or design ground motion at the ground surface of a site might be 0.30 g (0.30 times the acceleration due to gravity) corresponding to a 1000-year return period. Alternately stated, this same 0.30 g would have a 7 percent chance of being equaled or exceeded in 75 years at this site.

A 7 percent chance of something occurring over say the 75-year life of a bridge is perceived as more relevant on a human time scale than stating that the same effect could happen with a 1000-year return period. However, the two statements are equivalent. Thus, even though the return periods that are used in this document sound quite long, they are indeed relevant when considered as a statement of odds over a reasonable time of exposure for the SDOT bridge inventory.

The relationship between return period and the probability of exceedance for a given period of time, taken here as "life," is as follows.

$$\text{Return Period} = -\text{Life} / \ln(1 - \text{Probability of Exceedance})$$

and

$$\text{Probability of Exceedance} = 1 - e^{-(\text{Life} / \text{Return Period})}$$

where, Return Period and Life are in years, Probability of Exceedance is given as a fraction, and "ln" is the natural logarithm and "e" is the base for the natural logarithm, 2.71828…

Example: 475 years = -50 years / ln (1.00 – 0.10) or 10% probability of exceedance in 50 years

APPENDIX D

STATE OF SEISMIC RETROFIT TECHNOLOGY

APPENDIX D - STATE OF SEISMIC RETROFIT TECHNOLOGY

In recent years the technical capabilities and tools available to the bridge engineer have continued to increase in sophistication and number. Currently, one of the key tools for assessment of existing structures is the displacement focused pushover method, which compares displacement capacities relative to demand, and importantly, keeps track of the internal force resisting elements as damage (plastic hinging) occurs. Using an upper and lower level seismic event for analysis provides reasonable limits for retrofit strategies. This method has been widely used in recent years and is more appropriate for structures in high seismic areas, such as Seattle.

Beyond analytical tools, experience has been gained in implementation and construction of the more popular retrofit techniques, such as column jacketing. Significant numbers of bridges have been retrofitted with column jacketing in the Seattle area. WSDOT for example has installed such retrofits on more than 80 bridges with single column piers (Reference 7)

Caltrans has developed extensive experience in retrofitting their structures, including column jacketing, restrainers, base isolation, improved seats, and a myriad of other techniques. Caltrans along with other agencies, including WSDOT, also have invested heavily in technology development, including extensive laboratory proof testing of potential retrofit techniques. Much of this work began as a result of damage experienced in the 1989 Loma Prieta earthquake; thus the bulk of this work emerged concurrent with or after the City's retrofit program. The results of this research will help the City's current phase of retrofit.

APPENDIX E

STATUS OF OTHER SEISMIC RETROFIT EFFORTS

APPENDIX E - STATUS OF OTHER SEISMIC RETROFIT EFFORTS

Numerous owners around the country have undertaken seismic assessment and retrofit programs. Several of these include large inventory owners, such as Caltrans and WSDOT. Owners of such large and spread out inventories generally have taken a multiple-pass approach to implementing seismic retrofits and achieving their ultimate goal of improved seismic safety. Both Caltrans (Reference 8) and WSDOT (Reference 7) have used approaches of improving superstructures first, columns second and foundations third. Whilst such an incremental approach intends to address the most vulnerable features first, it is possible that a higher vulnerability is created by the partial retrofit. Although rare, the potential for creating such a problem should be considered by the engineer assessing a given structure, with the guiding principle to create no more harm than currently exists.

Local agency owned bridges in California are gradually being evaluated and retrofitted under the Local Bridge Seismic Safety Retrofit Program (Reference 8), and the lead agencies are Los Angeles County, Santa Clara County, and Caltrans, who is the lead for all local bridges except those in the two listed counties. The primary philosophy of this program is to prevent bridge collapse in the event of a maximum credible earthquake, which is consistent with Caltrans practice for its new bridges (Reference 9). Higher performance levels may be considered, although extra cost would be anticipated and this would be the responsibility of the local agency.

APPENDIX F

SEISMIC RETROFIT BRIDGE DATA SUMMARY GUIDELINES

APPENDIX F- SEISMIC RETROFIT BRIDGE DATA SUMMARY GUIDELINES

Seattle Department of Transportation
Bridge Seismic Retrofit Program - Phase III
Bridge Seismic Retrofit Design Criteria

Seismic Retrofit Bridge Data Summary

		BRIDGE NAME		**Notes**
1	**Determine Performance Level**			
	Importance			To be confirmed or revised by SDOT. Essential bridges are those that are expected to function after an earthquake. All other bridges are classified as standard.
2	**ASL (Anticipated Service Life)**	ASL 2		
	(After Rehab.)			To be confirmed or revised by SDOT.
3	**Performance Level:**	Lower Level Ground Motion, 50% exceedance in 75 years	Upper Level Ground Motion, 7% exceedance in 75 years	PL1 is life Safety (significant damage is sustained and service is significantly distrupted, but life saftey is preserved. The bridge may need to be replaced after a large earthquake.) PL2 is Operational (Damage sustained is minimal and service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be repairable with or without restrictions on traffic flow.) PL3 is Fully Operational (Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Damage is repairable without interruptions to traffic.)
	Return Period, years	100	1000	
		Spectrum Ordinates (g)	Frequency of Exceedence ($1/T_{return}$)	USGS 2008 Interactive Deaggregations website (geohazards.usgs.gov/deaggint/2008/)
	Seismic Hazard Ordinates, $S_{0.2}$			
	Seismic Hazard Ordinates, S_1			
4	**Determine Seismic Hazard Level**			USGS 2008 Interactive Deaggregations website (geohazards.usgs.gov/deaggint/2008/)
	Location, Zip Code, (Latitude, Longitude)			
	Spectral Acceleration			
	S_S			
	S_1			
	PGA			Results from the Hazard Calculation
	F_{PGA}			Table 1 (APPENDIX G)
	Soil Site Class (per Geotechnical Engineer)			To be provided by the Geotechnical Engineer
	Soil Factors, F_a			Table 1 (APPENDIX G)
	Soil Factors, F_v			Table 1 (APPENDIX G)
	Spectral Accelerations including Site Effects			
	$S_{DS} = F_a S_S$			Fig 4, to be verified by the Geotechnical Engineer
	$S_{D1} = F_v S_1$			Fig 4, to be verified by the Geotechnical Engineer
5	**SHL (Seismic Hazard Level)**			
6	**SRC (Seismic Retrofit Category)**			
7	**Screening: Components to Be Screened**			To preform preliminary assessment on these compoenents in data collection and conceptual seismic analysis phase, to be verified by SDOT
8	**Evaluation:**			
	Evaluation Method to be used			Select during the conceptual seismic analysis phase, to be verified by SDOT
9	**Retrofitting:**			
	Components to be retrofitted if deficient			Select during the retrofit schematic and post-retrofit conceptual seismic analysis phase, to be verified by SDOT.

Notes:
1. - YELLOW INDICATES FIELDS THAT REQUIRE INPUT INFORMATION BY THE ENGINEEF
2. "NOTES" COLUMN ABOVE PROVIDES GUIDELINES AND REFERENCES TO ASSIST THE ENGINEER IN FILLING OUT THE SEISMIC RETROFIT BRIDGE DATA SUMMARY SHEET

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Seismic Retrofit Bridge Summary_Blank-08-12-2008-RevA

APPENDIX F- SEISMIC RETROFIT BRIDGE DATA SUMMARY GUIDELINES

Seattle Department of Transportation
Bridge Seismic Retrofit Program - Phase III
Bridge Seismic Retrofit Design Criteria

Seismic Retrofit Bridge Data Summary

		BRIDGE NAME		**Notes**
1	**Determine Performance Level**			
	Importance			To be confirmed or revised by SDOT. Essential bridges are those that are expected to function after an earthquake. All other bridges are classified as standard.
2	**ASL (Anticipated Service Life)**	ASL 2		
	(After Rehab.)	(16-50 Yrs)		To be confirmed or revised by SDOT.
3	**Performance Level:**	Lower Level Ground Motion, 50% exceedance in 75 years	Upper Level Ground Motion, 7% exceedance in 75 years	PL1 is life Safety (significant damage is sustained and service is significantly distrupted, but life saftey is preserved. The bridge may need to be replaced after a large earthquake.) PL2 is Operational (Damage sustained is minimal and service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be repairable with or without restrictions on traffic flow.) PL3 is Fully Operational (Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Damage is repairable without interruptions to traffic.)
	Return Period, years	100	1000	
		PL3	PL1	
		Spectrum Ordinates (g)	Frequency of Exceedence ($1/T_{return}$)	USGS 2008 Interactive Deaggregations website (geohazards.usgs.gov/deaggint/2008/)
	Seismic Hazard Ordinates, $S_{0.2}$	0.288	0.011	
		0.432	0.005	
	Seismic Hazard Ordinates, S_1	0.0641	0.016	
		0.0961	0.009	
4	**Determine Seismic Hazard Level**			USGS 2008 Interactive Deaggregations website (geohazards.usgs.gov/deaggint/2008/)
	Location, Zip Code, (Latitude, Longitude)	98134, (47.577, -122.331)		
	Spectral Acceleration			
	S_s	0.307	1.131	
	S_1	0.091	0.323	
	PGA	0.142	0.42	Results from the Hazard Calculation
	F_{PGA}	2.22	0.9	Table 1 (APPENDIX G)
	Soil Site Class (per Geotechnical Engineer)	E	E	To be provided by the Geotechnical Engineer
	Soil Factors, F_a	1.88	0.9	Table 1 (APPENDIX G)
	Soil Factors, F_v	3.5	2.71	Table 1 (APPENDIX G)
	Spectral Accelerations Including Site Effects			
	$S_{DS} = F_a S_s$	0.578	1.018	Fig 4, to be verified by the Geotechnical Engineer
	$S_{D1} = F_v S_1$	0.317	0.875	Fig 4, to be verified by the Geotechnical Engineer
5	**SHL (Seismic Hazard Level)**	IV	IV	
6	**SRC (Seismic Retrofit Category)**	D	C	
7	**Screening: Components to Be Screened**	Seat Widths, Connections, Columns, Walls, Footings, Abutments, & Liquefaction	Seat Widths, Connections, Columns, Walls, Footings, Abutments, & Liquefaction	To preform preliminary assessment on these compoenents in data collection and conceptual seismic analysis phase, to be verified by SDOT
8	**Evaluation:**			
	Evaluation Method to be used	C: Component C/D method using elastic dynamic analysis methods	B/C/D1/D2: Capacity spectrum method: Structure C/D method (Displacement Capacity Evaluation)	Select during the conceptual seismic analysis phase, to be verified by SDOT
9	**Retrofitting:**			
	Components to be retrofitted if deficient	Seat and Connections; Columns, Walls, Footings, Abutments & Liquefaction	Seat and Connections; Columns, Walls, Footings, Abutments & Liquefaction	Select during the retrofit schematic and post-retrofit conceptual seismic analysis phase, to be verified by SDOT.

Example Sheet
Seismic Retrofit Bridge Summary_Blank-08-12-2008-RevA

APPENDIX G

DETERMINATION OF DESIGN RESPONSE SPECTRA

APPENDIX G - DETERMINATION OF DESIGN RESPONSE SPECTRA

G.1 Introduction

This appendix describes the methodology that should be used to determine design response spectra for earthquakes with a 100-yr[1] return period (Lower Level Earthquake or LLE) or a 1,000-yr return period (Upper Level Earthquake or ULE), consistent with requirements of SDOT's *Bridge Seismic Retrofit Philosophy, Policies, and Criteria*. The methodology described in this appendix uses information that is obtained from either the United States Geological Survey (USGS) seismic hazard curve application tool or the USGS Interactive Deaggregation, both found on the USGS website. With the exception of the methods described in this appendix, the development of the design response spectra is consistent with all other procedures outlined in the 2nd edition of the AASHTO *Guide Specifications for Seismic Bridge Design* (Guide Specifications).

G.1.1 Recommended Seismic Ground Motion Model

Procedures in the AASHTO Guide Specifications currently use a seismic model that was developed by the USGS in 2002. This seismic model was based on the then current scientific thinking on seismic sources that could result in ground shaking at any location within continental United States, as well as how these ground motions propagate from the seismic source to a site of interest. The USGS updated the 2002 seismic model in 2008 and again in 2014. These updates account for new scientific thinking regarding the source of ground shaking and how the ground motions propagate through the crust of the earth to the ground surface. In the Seattle area, the updates resulted in small reductions in the design spectral acceleration ordinates.

With the original adoption of the 2002 USGS seismic model, AASHTO decided that it was not going to necessarily update its seismic model each time that USGS published a new seismic model for the United States. Rather AASHTO wanted to maintain more stability to their seismic design process. By doing so, AASHTO also understood that it would not necessarily be using the latest thinking in seismic source modeling and GMPEs.

Now that the 2002 USGS seismic model is over 10 years old, many transportation agencies, including WSDOT and SDOT, are allowing use of the 2008 USGS seismic model for the determination of design response spectra, subject to the review and approval of the transportation agency. To use the 2008 USGS seismic model, it is necessary to use either the 2008 USGS hazard website tool or the 2008 USGS Interactive Deaggregation tool within the USGS website, rather than the current AASHTO website tool, since the current AASHTO website tool is based on the 2002 USGS seismic model.

G.1.2 Selection of Design Spectra

This document describes how the 2008 USGS hazard website tool or the 2008 USGS Interactive Deaggregation website tool can be used to develop the design response spectra for SDOT retrofit projects. This methodology is recommended over the current (2002) AASHTO seismic model, because it is more consistent with current scientific knowledge regarding seismic ground motions in the Seattle area.

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[1] Nominal return periods of 1,000 years and 100 years will be used to define the Upper Level Earthquake (ULE) and the Lower Level Earthquake (LLE), respectively. These return periods are used to define earthquakes with probabilities of exceedance of 7% in 75 years and 50% in 75 years. If calculated using equations in Appendix C, the return periods are more precisely defined as 1,033 years and 108 years for the ULR and LLE, respectively. This distinction becomes important when using some of the tools within the USGS website. For the ULE the USGS website tool using 5% probability of exceedance in 50 years is very similar to the 7% in 75 year exceedance used by AASHTO. However for the LLE the difference between a return period of 100 years and 108 years is larger, and therefore, while the nominal return period is 100 years for the LLE, the 108-yr return period must be used in the USGS hazard website to obtain appropriate PGA and spectral accelerations.

The methodology is also easily adaptable to new 2014 USGS ground motion maps, once they become available either later this year or next.

When applying the 2008 USGS website tool, the resulting design spectra for the nominal 1,000-yr return period should be checked against the design spectra obtained with the current AASHTO website tool. In general slight reductions in spectral ordinates should be seen for firm-ground (i.e., AASHTO Site Class B) conditions when comparing design spectra developed using the 2008 USGS seismic model and design spectra from the older seismic model used by the AASHTO website tool. If the difference in spectral ordinate is more than 10% for any period, the designer should bring the difference to the attention of SDOT to determine if the design spectra from the older AASHTO website tool should be used.

G.1.3 Use of Firm-Ground Motions

The 2008 USGS website tool allows the user to define Vs30 [i.e., time averaged shear wave velocity in the upper 30 m (100 ft)]. This procedure in intended to provide ground motions at the ground surface, and thereby, avoid the need to adjust firm-ground motions by short-period (Fa) and long-period (Fv) site adjustment factors. The newer direct method does not result in the same ground surface response spectra, as shown by Stewart and Seyham (2013). A task force has been working for the last several years to reconcile the two approaches. This work will result in changes in Fa and Fv that are more consistent with the direct determination method in the 2008 USGS website tool.

The Provision Update Committee (PUC) of the Building Seismic Safety Council (BSSC) is currently updating values of Fa and Fv in the International Building Code (IBC). The new Fa and Fv factors will be published in the next edition of ASCE 7 *Minimum Design Loads for Buildings and Other Structures* – to be published in 2016. Any changes in Fa and Fv will be in conflict with those in the current AASHTO Guide Specifications.

In view of these pending changes and since new Fa and Fv have not been published, the approach recommended for SDOT retrofit projects is to determine ground motions for firm-ground conditions (i.e., Site Class B or Vs30 = 760 m/sec) and then make adjustments to the firm-ground response spectrum using Fa and Fv published in the AASHTO Guide Specifications. This approach maintains more consistency with the current AASHTO Guide Specifications. It also allows easier adjustment for near-fault directivity effects at the firm-ground level.

G.1.4 Use of Site-Specific Modeling

As an alternative to the general method for design spectra determination described in this appendix, AASHTO allows use of site-specific procedures to determine firm-ground response spectra and the change in ground motions from near-surface geology. These methods are permitted by SDOT on a site-specific basis subject to SDOT's approval. Procedures for conducting these site-specific studies should follow methods in the current AASHTO Guide Specification.

A number of probabilistic seismic hazard analyses (PSHA) have been conducted in the Seattle area over the past 10 years to establish firm-ground spectra. These PSHAs have been conducted for the Alaska Way Viaduct project, the Seattle Sea Wall project, and for some high-rise structures in downtown Seattle. Results of these site-specific PSHAs can serve is a sufficient basis for establishing firm-ground design spectra with either a nominal 1,000-yr or 100-yr return period, as long as they were conducted in the City of Seattle and they are newer than the 2008 USGS seismic model, subject to SDOT's approval.

As part of the approval process, SDOT will require a detailed review of the PSHA and may require an independent third-party review to confirm that methodologies used to conduct the site-specific PSHA is consistent with the latest state-of-the practice. Other requirements in the AASHTO Guide Specification, including use of a maximum one-third reduction in the spectral acceleration ordinates, apply to any site-specific PSHA evaluation.

G.2 Design Spectra for USGS Hazard Website Tool

Steps required to determine design response spectra for the LLE and ULE on a SDOT retrofit project are listed below. In this application the USGS hazard website is used. This approach is generally preferred as the USGS hazard website tool provides plotted response spectra. This plot is helpful in evaluating the quality of the resulting spectra, and therefore can serve as an early warning of incorrect spectra determination.

Step 1: Determine latitude and longitude of project site. This information can be obtained from Google Earth on other similar systems. For this discussion, the Seattle Municipal Tower (SMT) will be used (47.6051; -122.3296) as an example location and application of the recommended approach.

Step 2: Open the USGS hazard website at the following link:

http://geohazards.usgs.gov/hazardtool/application.php

Enter the coordinates for the site in the dialogue box to set the project location. Once the latitude and longitude are loaded, hit the arrow on the right side of the box.

[NTD: At the time that this appendix was written, the location box did not seem to consistently accept the negative longitude value. To get around this problem the latitude and longitude can be copied and pasted from a Word file. Be sure to use the negative sign in front of the longitude.]

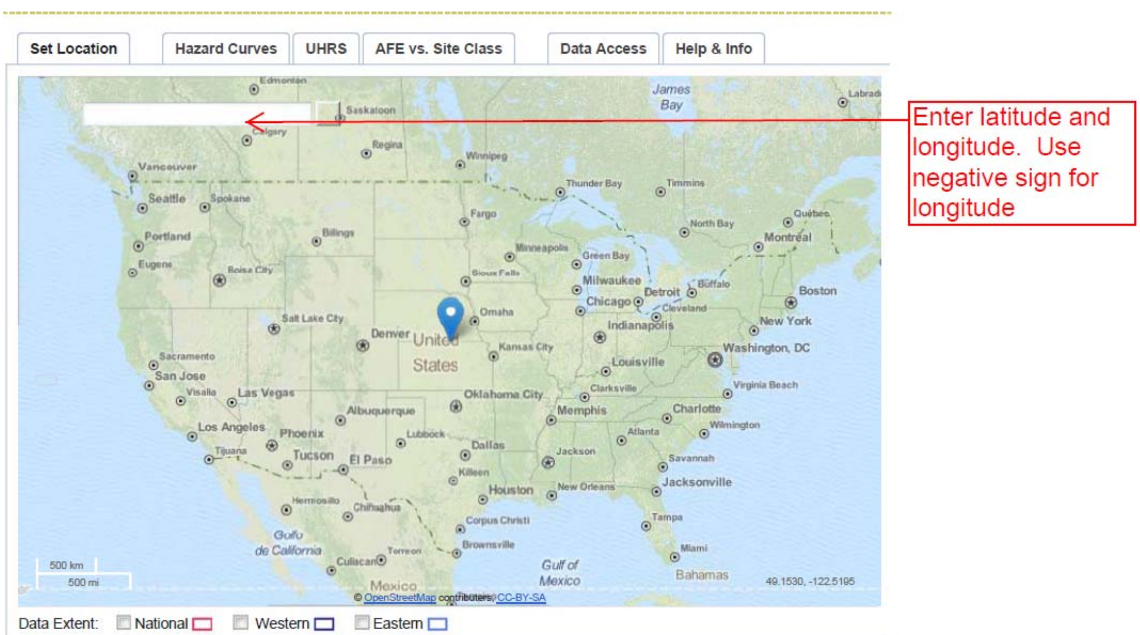

Figure 1. USGS website for estimating response spectra at locations within the United States.

Step 3: Click on the tab called UHRS. This tab will provide the uniform hazard response spectra (UHRS) for probabilities of exceedance of 2, 5, 10, and 20% in 50 yrs. These return periods correspond to approximately 2,475, 975, 475, and 225 yrs, respectively. Figure 2 shows the hazard spectra obtained for the SMT site.

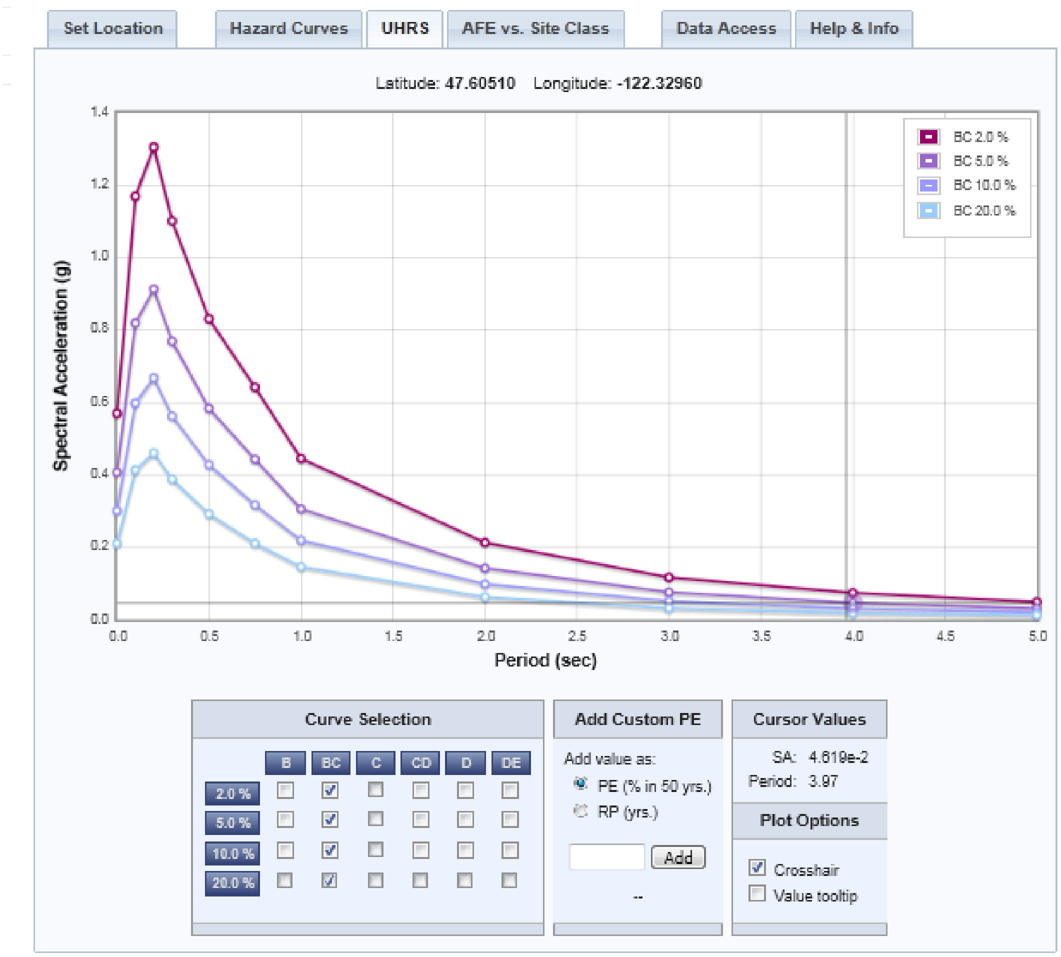

Figure 2. Ground motion hazard spectra for B/C site conditions at the SMT site.

Note that the B/C boxes are used rather than the B boxes. The B/C box provides a firm-ground response spectra consistent with Site Class B in the AASHTO Guide Specifications. The average Vs30 for "Curve Selection B" is 1150 m/sec (3,772 ft/sec) and therefore inconsistent with the AASHTO Guide Specification for Site Class B.

The return period can also be specified in the box "Add Custom PE". Since we are interested in the 100 and 1,000 yr return periods, activate the RP box and, for this example, input 108, meaning that the tool should define the hazard response spectrum for 108 yrs. This will add the response spectrum for a 108-yr return period to the graph above.

Boxes for 2, 10, and 20% exceedance can be turned off by clicking on the boxes for 2, 10, and 20% to the left. These spectra are for the 2,475-, 475-, and 225-yr return periods and do not require evaluation by SDOT. This leaves two hazards curves, one representative of the nominal 1,000-yr earthquake and the other that is for the nominal 100-yr earthquake return period. The resulting plot is shown in Figure 3.

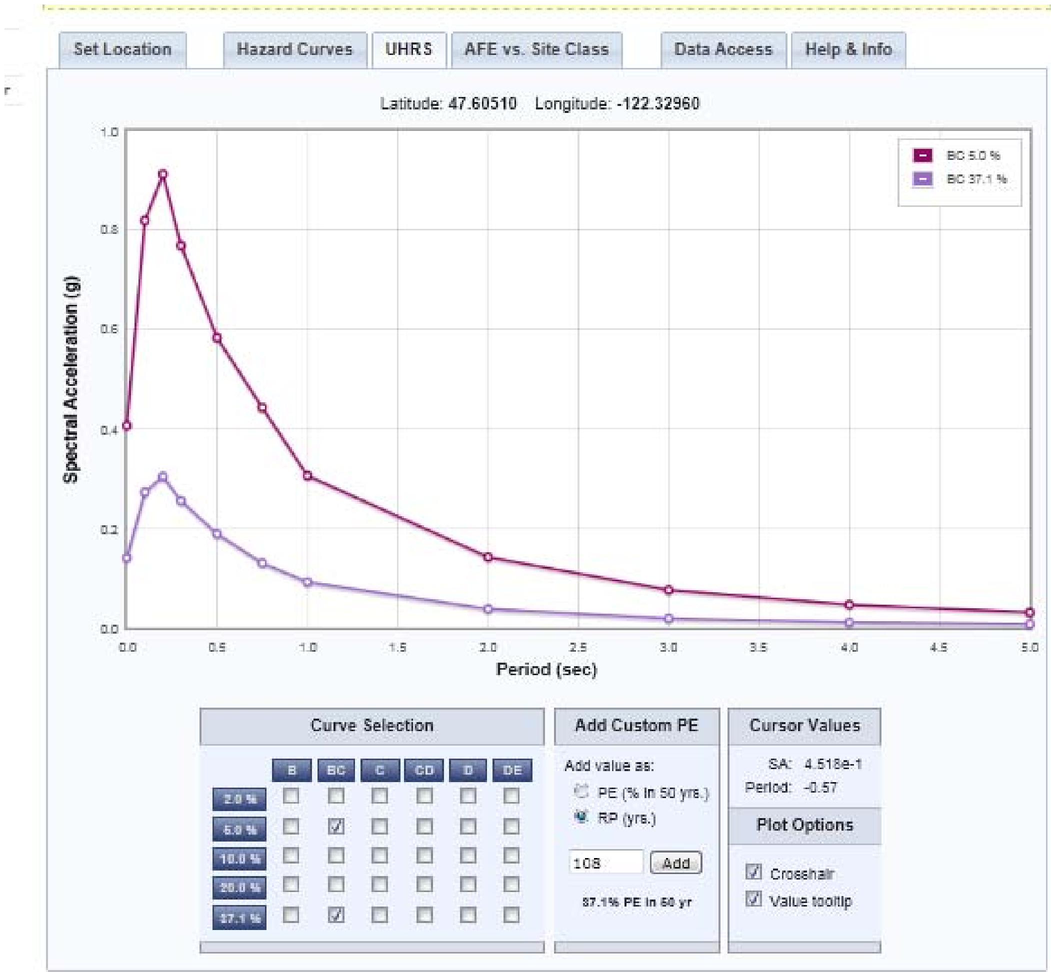

Figure 3. Hazard Curves for SMT site at nominal 1,000-yr and 100-yr earthquakes.

Values for the points on the resulting plot can be obtained by checking the box “Value Tooltip” and hovering over the values of PGA and spectral acceleration.

[NTD: The top of the column of boxes identifies the site class. The Help & Info menu identifies the Vs30 values for each of these site classes. Comparisons were made between the AASHTO website results for PGA, Ss, and S1 for different site classes and the PGA, Ss, and S1 for the 1,000-yr earthquake. In general the comparison for the SMT site was relatively close for AASHTO Site Classes B, C, and D when compared to USGS hazard values for Site Classes B/C, C, and D. However, Site Class D/E from the USGS hazard website is significantly higher than the AASHTO values. As noted in the Help & Info menu in the hazard site, the Vs30 for Site Class D/E is 180 m/sec (590 ft/sec) which compares to the Vs30 range in AASHTO for Site Class E of < 600 fps. According to Stewart and Ceyhan (2013) the midpoint or median of the range for Site Class E, which is the same as Site Class E in the AASHTO classification, ranges from approximately 150 to 155 m/sec (492 to 508 ft/sec). This difference in velocity likely contributes to the difference for Site Class E. The conclusion from this comparison is that when using the USGS hazard curve, the website tool for different site classes could be used to estimate PGA, Ss, and S1 for Site Classes B, C, and D but the hazard website tool should not be used for Site Class D/E to represent Site Class E, as spectral accelerations are substantially higher for Site Class D/E.]

Step 4: Determine design spectra using PGA, Ss, and S1 results from the hazards calculation. The design spectrum for the nominal 1,000-and 100-yr return period are obtained by using the spectral accelerations at 0.2 and 1.0 seconds following the design spectrum construction method in Section 3.4.1 of the AASHTO Guide Specification. The spectra shown in Figure 3 represents firm-ground conditions (B/C Boundary or Site Class B based on AASHTO terminology) where the shear wave velocity is equal to 760 m/sec (~2,500 fps).

[NTD: If the project site is located with 6 miles of the Seattle Fault on the Southern Whidbey Island Fault, the firm-ground spectrum must be adjusted for near-fault effects. Simplified Caltrans procedures can be used to make this adjustment. In general spectral accelerations less than 0.5 second require no changes. Spectral accelerations at 1.0 seconds and above should be increased by 20%. Between 0.5 and 1.0 seconds a linear increase between 0 and 20% should be used. While this procedure lacks some of the rigor of newer, more complete methods, it is simple to apply and is believe to result in representative adjustments.]

Step 5: Develop design spectra at the ground surface for nominal return periods of 1,000 and 100 years from the hazard spectra established in Step 4. This step involves adjusting the B/C hazard spectral ordinates for local site conditions and then using the resulting three anchor points (i.e., $A_S = F_{pga}PGA$, $S_{DS} = F_aSs$, and $S_{D1} = F_vS1$) to develop the design spectra. Either standard site-adjustment factors given below or site-specific ground response analyses can be used to account for local site effects.

If site-specific procedures are used to determine F_{pga}, F_a, and F_v, then guidance in Section 3.4.3.2 of the AASHTO Guide Specifications should be followed when modeling the soil profile, selecting earthquake records, and conducting earthquake response analyses.

Table 3.4.2.3-1—Values of F_{pga} and F_a as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	$PGA \leq 0.10$ $S_s \leq 0.25$	$PGA = 0.20$ $S_s = 0.50$	$PGA = 0.30$ $S_s = 0.75$	$PGA = 0.40$ $S_s = 1.00$	$PGA \geq 0.50$ $S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

Note: Use straight line interpolation for intermediate values of PGA and S_s, where PGA is the peak ground acceleration and S_s is the spectral acceleration coefficient at 0.2 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

Note: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration coefficient at 1.0 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

Table 1 - Summary of Site Coefficients from AASHTO Guide

Specifications

The form of the response spectra for the three-point method is shown in Figure 4 below. Equations from the AASHTO Guide Specifications for determining S_a (below T_o), T_o, and T_s are summarized in the equations following Figure 4.

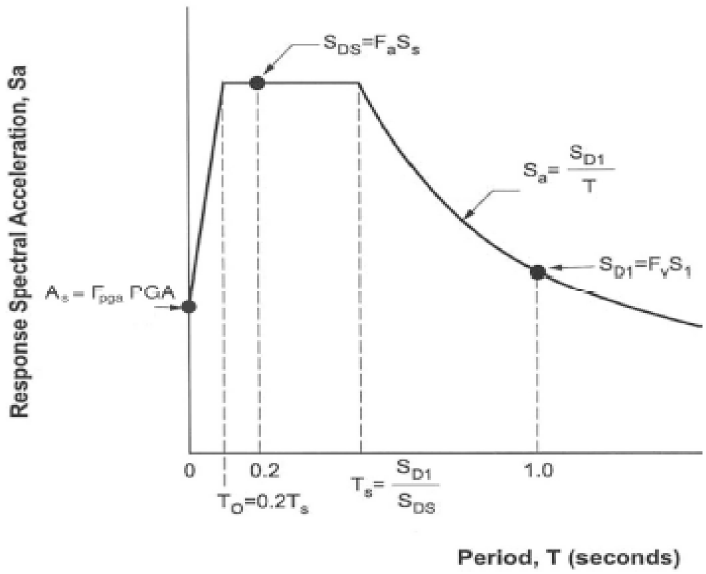

Figure 4. Design spectrum as defined in Section 3.4.3.2 of the AASHTO Guide Specifications.

$$S_a = (S_{DS} - A_s)\frac{T}{T_o} + A_s \qquad (3.4.1\text{-}4)$$ and

in which:

$$T_o = 0.2T_S \qquad (3.4.1\text{-}5)$$

$$T_S = \frac{S_{D1}}{S_{DS}} \qquad (3.4.1\text{-}6)$$

where:

A_s = acceleration coefficient

S_{D1} = design spectral acceleration coefficient at 1.0-sec period

S_{DS} = design spectral acceleration coefficient at 0.2-sec period

T = period of vibration (sec)

- For periods greater than or equal to T_o and less than or equal to T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = S_{DS} \qquad (3.4.1\text{-}7)$$

- For periods greater than T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = \frac{S_{D1}}{T} \qquad (3.4.1\text{-}8)$$

For the Seattle Municipal Tower location, the firm-ground (Site Class B) values for PGA, Ss, and S1 are summarized in the table below for the nominal 1,000- and 100-yr earthquakes.

SMT Site	Nominal Earthquake Return Period (years)	
	1,000	100
PGA and Spectral Accelerations for B/C Boundary		
PGA	0.45g	0.14g
Ss	1.00g	0.31g
S1	0.33g	0.09g
Site Factors (Assumed for Example to be Site Class D)		
Fpga	1.05	1.52
Fa	1.1	1.55
Fc	1.74	2.4
Site-Adjusted Design Values		
A_S	0.47	0.21
F_{DS}	1.1	0.48
F_{D1}	0.57	0.22

Table 2 - PGA and spectral accelerations for example SMT site.

Step 6: Use AASHTO three-point method to develop design spectra for the site. The design spectra should not be carried beyond a period of 5 seconds without special studies. Figure 5 shows the design response spectra developed for the nominal 1,000- and 100-yr earthquakes for the SMT site.

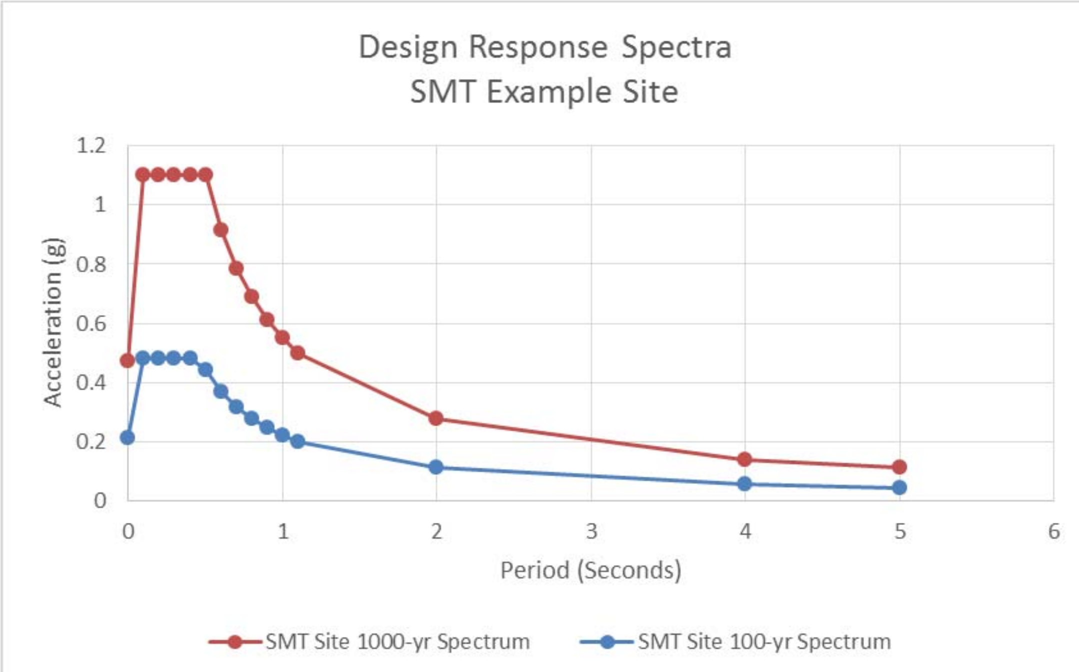

Figure 5. Plot of nominal 1,000- and 100-yr spectra for SMT example.

G.3 Design Spectra from USGS Interactive Deaggregation Website Tool

The second approach that can be used to determine the design response spectra for the ULE and LLE follows many of the same steps described above using the USGS hazard website tool. However, rather than using the hazard website, the interactive deaggregation website is used. The link for this website is provided below:

http://geohazards.usgs.gov/deaggint/2008/

The advantage of this site is that spectral ordinates can be determined for the 2002 and 2008 USGS seismic hazard models. If the desire of the designer or SDOT is to maintain consistency with the maps used in the AASHTO Guide Specification, then interactive deaggregation website can be used. However, this website provides a significant amount of data regarding sources of ground motions and therefore may create some questions on how the additional data should be used. Comparisons between the USGS hazard website tool and the interactive deaggregation result in some minor variations in PGA, Sa, and S1; however, these variations are generally not enough to warrant use of one method versus the other.

Steps 1 and 2: These steps are much like those described above. Figure 6 shows the data entry table for the 2008 interactive website.

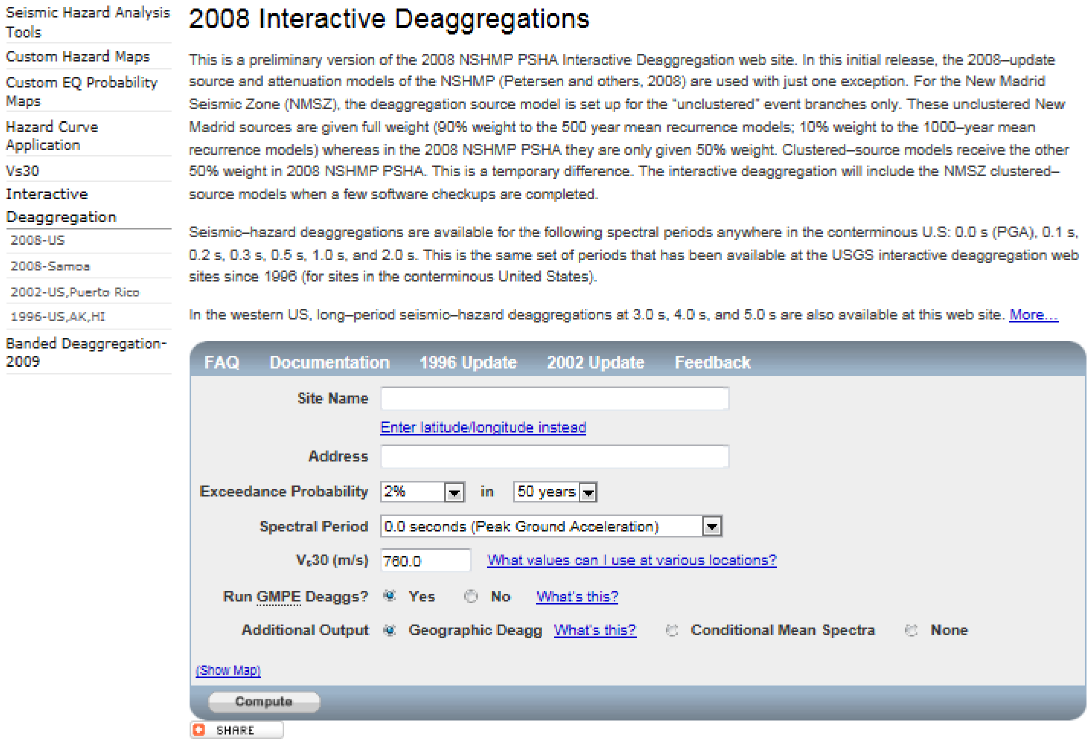

Figure 6. Input information of USGS Interactive Deaggregation website.

When setting up the evaluation, the latitude and longitude option should be used. The exceedance probability should be set to 5% in 50 yrs for the nominal 1,000-yr earthquake. For the nominal 100-yr earthquake, use 50% in 75 years. Spectra should then be determined from the website tool for spectral periods of 0, 0.2, and 1.0 seconds. The example below shows input for the nominal 1,000-yr earthquake. In this case the PGA will be computed. Results of the analyses are shown either graphically within the pdf or GIF file; tabulations of the data from the deaggregation is provided in the TXT file.

[NTD: Note that Vs30 values can be assigned within this website tool. The Vs30 values allow a site-adjusted ground motion to be calculated. This approach was developed as part of the NGA West 1 and 2 program. Although the method has not been formally adopted by AASHTO, it is being used as part of the International Building Code (IBC). Use of Fa and Fv for adjusting the firm-ground (Site Class B) and use of Vs30 for the site will likely not provide the same ground surface spectra. IBC is currently updating Fa and Fv values for ASCE 7-16 to make the two approaches more consistent. Until this update occurs, the recommended approach is to compute firm-ground motions by assuming the Vs30 = 760 m/sec.]

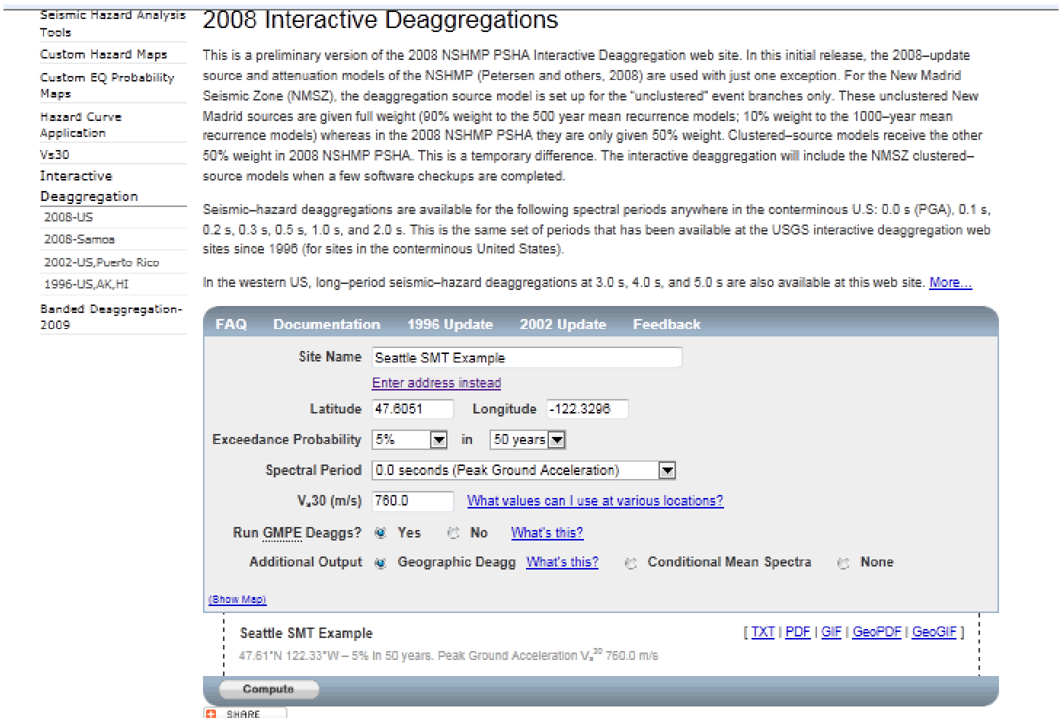

Figure 7. Interactive deaggregation input example for SMT site.

A screen shot showing part of the data from the interactive website is below in Figure 8. The PGA for the site is shown as 0.4110. This process is repeated for spectral accelerations of 0.2 and 1.0 seconds.

```
*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Seattle_SMT_Exa long: 122.330 W., lat: 47.605 N.
 Vs30(m/s)= 760.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08  See USGS OFR 2008-1128. dM=0.2 below
Return period: 975   yrs. Exceedance PGA =0.4110   g. Weight * Computed_Rate_Ex 0.102E-02
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00707
#This deaggregation corresponds to Mean Hazard w/all GMPEs
DIST(KM) MAG(MW) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2<EPS<-1 EPS<-2
    6.6    5.05    0.856    0.271    0.516    0.068    0.000    0.000    0.000
   12.9    5.05    0.145    0.145    0.000    0.000    0.000    0.000    0.000
   53.6    5.05    0.177    0.177    0.000    0.000    0.000    0.000    0.000
   62.4    5.05    0.075    0.075    0.000    0.000    0.000    0.000    0.000
    6.7    5.20    1.771    0.454    1.112    0.204    0.000    0.000    0.000
   13.1    5.20    0.392    0.385    0.007    0.000    0.000    0.000    0.000
   53.6    5.20    0.437    0.437    0.000    0.000    0.000    0.000    0.000
   63.0    5.21    0.236    0.236    0.000    0.000    0.000    0.000    0.000
    6.8    5.40    1.784    0.343    1.091    0.350    0.000    0.000    0.000
   13.3    5.40    0.544    0.479    0.064    0.000    0.000    0.000    0.000
   53.7    5.40    0.554    0.554    0.000    0.000    0.000    0.000    0.000
   63.4    5.40    0.359    0.359    0.000    0.000    0.000    0.000    0.000
   74.5    5.41    0.067    0.067    0.000    0.000    0.000    0.000    0.000
    6.9    5.60    1.646    0.240    0.937    0.469    0.000    0.000    0.000
   13.4    5.60    0.650    0.486    0.164    0.000    0.000    0.000    0.000
   53.8    5.60    0.672    0.672    0.000    0.000    0.000    0.000    0.000
```

Figure 8. Typical results from TXT file for interactive deaggregation evaluation.

Values of PGA, Ss, and S1 for Site Class B with a nominal return period of 1,000 yrs are 0.4110, 0.9226, and 0.3097g. For the nominal 100-yr return period the values of PGA, Ss, and S1 for Site Class B would be 0.1418, 0.3065, and 0.0923g, respectively. For practical purposes these spectral ordinates are the same as those listed in Table 2 determined using the USGS hazard website tool.

Steps 3 through 6: Follow steps described in the preceding section to define design response spectra based on the three-point method.

G.4 Vertical Response Spectra Determination

Neither the second edition of the AASHTO *Guide Specifications for LRFD Seismic Bridge Design* nor the 7th edition of the AASHTO *LRFD Bridge Design Specifications* provide any direction on the development of vertical spectra.

For projects that require a vertical response spectra, one acceptable approach for developing the vertical spectra is the methodology identified in Chapter 23 “Vertical Ground Motions for Seismic Design” of the FEMA P-750 report *NEHRP Recommended Seismic Provisions for Buildings and Other Structures* (FEMA, 2009). This methodology is summarized in Table 3 below. The approach replaces the former assumption that the ratio of the vertical to horizontal (V:H) response spectral ordinates at each period can be approximated by multiplying the horizontal spectra ordinate by 2/3rd. As pointed out in the NEHRP commentary to Chapter 23, the 2/3rd ratio can be conservative or unconservative depending on a number of factors, including spectral period, distance from the earthquake, local site conditions, and magnitude. Additional information about this approach can be found in FEMA, 2009 or publications by Campbell and Bozorgnia (2008).

Other methods for developing vertical response spectra from the horizontal spectra have been developed and may be suitable for use on projects requiring a vertical spectrum. Consult with SDOT on the approach to be implemented before carrying out the vertical spectra determination.

VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

23.1 DESIGN VERTICAL RESPONSE SPECTRUM. Where a design vertical response spectrum is required by these *Provisions* and site-specific procedures are not used, the design vertical response spectral acceleration, S_{av}, (in g – gravity unit) shall be developed as follows:

1. For vertical periods less than or equal to 0.025 second, S_{av} shall be determined in accordance with Equation 23.1-1 as follows:

$$S_{av} = 0.3C_V S_{DS} \qquad (23.1\text{-}1)$$

2. For vertical periods greater than 0.025 second and less than or equal to 0.05 second, S_{av} shall be determined in accordance with Equation 23.1-2 as follows:

$$S_{av} = 20C_V S_{DS}(T_V - 0.025) + 0.3C_V S_{DS} \qquad (23.1\text{-}2)$$

3. For vertical periods greater than 0.05 second and less than or equal to 0.15 second, S_{av} shall be determined in accordance with Equation 23.1-3 as follows:

$$S_{av} = 0.8C_V S_{DS} \qquad (23.1\text{-}3)$$

4. For vertical periods greater than 0.15 second and less than or equal to 2.0 seconds, S_{av} shall be determined in accordance with Equation 23.1-4 as follows:

$$S_{av} = 0.8C_V S_{DS}\left(\frac{0.15}{T_V}\right)^{0.75} \qquad (23.1\text{-}4)$$

where C_V is defined in terms of S_S in Table 23.1-1, S_{DS} = the design spectral response acceleration parameter at short periods, and T_V = the vertical period of vibration.

Table 23.1-1 Values of Vertical Coefficient C_V

MCE$_R$ Spectral Response Parameter at Short Periods [a]	Site Class A, B	Site Class C	Site Class D, E, F
$S_S \geq 2.0$	0.9	1.3	1.5
$S_S = 1.0$	0.9	1.1	1.3
$S_S = 0.6$	0.9	1.0	1.1
$S_S = 0.3$	0.8	0.8	0.9
$S_S \leq 0.2$	0.7	0.7	0.7

[a] Use straight-line interpolation for intermediate values of S_S.

S_{av} shall not be less than one-half (1/2) of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

For vertical periods greater than 2.0 seconds, S_{av} shall be developed from a site-specific procedure; however, the resulting ordinate of S_{av} shall not be less than one-half (1/2) of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

In lieu of using the above procedure, a site-specific study may be performed to obtain S_{av} at vertical periods less than or equal to 2.0 seconds, but the value so determined shall not be less than 80 percent of the S_{av} value determined from Equations 23.1-1 through 23.1-4.

Table 3 - Methodology for determining vertical response spectra ordinates

G.5 References

AASHTO LRFD

AASHTO Guide Specification

International Building Code

ASCE 7-10

NEHRP P-750

Stewart and Seyham