I-5 Study

Seismic Report and Funding Request (SSB 5975, Section 209)

December 1, 2022

Americans with Disabilities Act (ADA) Information: This material can be made available in an alternate format by emailing the Office of Equal and Civil Rights at wsdotada@wsdot.wa.gov or by calling toll free, 855-362-4ADA (4232). Persons who are deaf or hard of hearing may make a request by calling the Washington State Relay at 711.

Title VI Notice to Public: It is the Washington State Department of Transportation's (WSDOT) policy to assure that no person shall, on the grounds of race, color, national origin or sex, as provided by Title VI of the Civil Rights Act of 1964, be excluded from participation in, be denied the benefits of, or be otherwise discriminated against under any of its programs and activities. Any person who believes his/her Title VI protection has been violated, may file a complaint with WSDOT's Office of Equal and Civil Rights. For additional information regarding Title VI complaint procedures and/or information regarding our non-discrimination obligations, please contact the Office of Equal and Civil Rights' Title VI Coordinator at (360) 705-7090.

Opening Letter

Dear Joint Transportation Committee Members

This spring, the Washington State Legislature passed the historic Move Ahead Washington transportation package which included $40 million intended to create a comprehensive I-5 master plan and a modern vision for the statewide corridor.

As an initial requirement, SSB 5975, Section 209 directed WSDOT to submit a recommended approach and funding request to:

- Assess the seismic risk of the I-5 causeway from Boeing Field to Lake City Way
- Recommend future work to mitigate seismic risk on the causeway, including estimated costs

WSDOT is pleased to submit the attached report presenting a recommended approach for future seismic mitigation for over 150 structures between Boeing Field and Lake City Way, in the Puget Sound region.

Additional work underway as part of the I-5 Corridor Study
WSDOT is continuing work on two other I-5 efforts required by the proviso, each due to the legislature on June 30, 2023:

- **HOV State-wide Performance**: Develops near term recommendations for improving the HOV system and evaluates other techniques for making the HOV system more efficient.
- **I-5 Corridor Planning**: Approximately 70 listening sessions are underway with key jurisdictions and other stakeholders statewide through early 2023. This feedback will inform a report recommending future planning efforts that will create a modern vision for the corridor.

Funding Request
WSDOT requests **$11.9 million** in the 2023-2025 biennium to advance this I-5 corridor work, specifically:

<u>Seismic Resiliency (Boeing Field to Lake City Way)</u>
The following steps will provide information needed to refine the level of effort and develop informed cost estimates for the Seismic Vulnerability Analysis, a critical first phase that creates a solid understanding of each bridge's earthquake resilience.

- **Lifeline Designation:** Determine if the study corridor should be included as part of a designated lifeline route and pursue next steps based on the designation.
- **Packaging and Phasing**: Develop and recommend packages of structures and phasing sequences to conduct the Seismic Vulnerability Analysis. Advance priority package(s) into the analysis.

<u>HOV System-Wide Performance</u>

- **Strategy and Initial Implementation**: Building on the 2023 legislative HOV performance recommendations, identify a pilot project to improve near-term system efficiency that progresses innovative and emerging technologies. Develop a project implementation plan and cost estimate and advance initial steps to launch the pilot project.

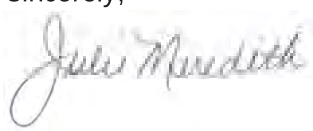
I-5 Corridor Planning

- **Progress Planning Work:** Building on the 2022-2023 listening sessions and legislative recommendations, develop a framework, coordinate corridor needs, and develop core evaluation criteria and a prioritization process for an overall I-5 Master Plan, setting a vision for a resilient statewide system that is safe, sound and smart. This work will explore emerging technologies and include an equitable and transparent decision-making process and community and stakeholder engagement program.
- **Early Action Projects**: Identify early action priority projects that address safety and/or resiliency along the corridor.

Next Steps

We thank the Legislature for their support in developing a modern vision for the I-5 corridor, a vital economic corridor in the state of Washington and the west coast. We look forward to continuing to work with the Governor's Office and the Legislature on next steps to deliver this vision.

Sincerely,



Julie Meredith, PE

Assistant Secretary, UMA and Megaprograms, Washington State Department of Transportation

Table of Contents

Table of Figures

Table of Tables

List of Abbreviations

3D .... three-dimensional
AASHTO .... American Association of State Highway and Transportation Officials
ADT .... average daily traffic
BDM .... Bridge Design Manual
C/D .... capacity demand
CIP .... cast in place
CQC .... Complete Quadratic Combination
DNR .... (Washington State) Department of Natural Resources
ECA .... Environmentally Critical Area
ERS .... earthquake-resisting system
FEE .... Functional Evaluation Earthquake
FHWA .... Federal Highway Administration
GIS .... geographic information system
HOV .... high-occupancy vehicle
I-5 .... Interstate 5
I-90 .... Interstate 90
I-405 .... Interstate 405
LiDAR .... light detection and ranging
LOE .... level of effort
MOT .... maintenance of traffic
NLTH .... nonlinear time history
PGA .... peak ground acceleration
QA/QC .... quality assurance and quality control
ROM .... rough order of magnitude
RSA .... response spectrum analysis
SCC .... Seattle Convention Center

SEE .... Safety Evaluation Earthquake
SR .... State Route
SSB .... Substitute Senate Bill
SSI .... soil-structure interaction
USGS .... United States Geological Survey
WSDOT .... Washington State Department of Transportation

Table of Appendices

Executive Summary

Study Purpose and Need

In spring 2022, the Move Ahead Washington funding package directed the Washington State Department of Transportation (WSDOT) to:

- Develop a recommended approach and funding request to assess the seismic risk of the I-5 causeway from Boeing Field to Lake City Way; and provide recommendations for future work to mitigate seismic risk on the causeway, including estimated costs.

Study Corridor Overview

Figure 1 shows the limits of the study area and the Interstate 5 (I-5) corridor from the Boeing Access Road at the south end to Lake City Way at the north end.

The study corridor includes 123 bridges and more than 50 earth-retaining walls or embankments. For the purposes of this study, the number of each classification is shown in Figure 2.

Figure 1: I-5 Seismic Study Area

Figure 2: Structures and Embankments—by the Numbers

Study Recommendations

Typically, once the planning steps are completed, there are three phases in a seismic upgrade process that build on one another to clearly define next steps and associated costs.

This study conducted a seismic risk assessment to recommend the type of Seismic Vulnerability Analysis for each study structure, associated level of effort (LOE), and a funding request. The funding request includes next-step efforts to advance seismic resilience and continue high-occupancy vehicle (HOV) efficiency and corridor planning work initiated by Substitute Senate Bill (SSB) 5975.

Figure 3 represents the overall recommended approach moving forward.

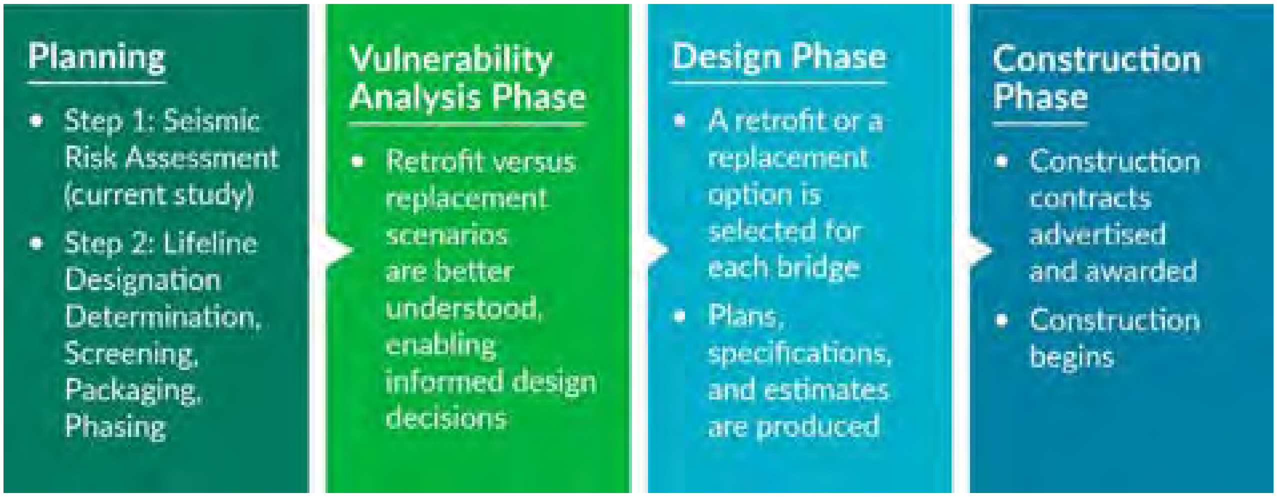


Figure 3: Recommended Program Phases

Performance Criteria

Figure 4: Current WSDOT Lifeline Map Showing the I-5 Seismic Study Area

This effort aligns with WSDOT's 2022 strategic plan, which seeks to improve the seismic resilience of the transportation system—notably by prioritizing and strengthening the elements of the transportation system most critical to emergency response after a seismic event. WSDOT has designated Interstate 405 (I-405) and much of I-5 and Interstate 90 (I-90) as Lifeline routes in the central Puget Sound area. The study segment of I-5 from Boeing Field to Lake City Way is not currently part of a designated Lifeline route. However, WSDOT's recently adopted resilience goal and the critical role this segment plays in moving people and goods in the region led the study to plan for a quick recovery of the area following a seismic event.

Historically, WSDOT has implemented "life-safety" retrofits for existing bridges on the Lifeline routes indicated in Figure 1 above. "Life-safety" emphasizes collapse prevention and these bridges may require significant repairs or even replacement after a large seismic event. Typically, seismic retrofits completed to date have not included foundation retrofits, which were held for future phases as funding is made available. For the purposes of this report, higher performance criteria were applied that exceed "life-safety" to evaluate bridges for resilience in a significant earthquake. This higher standard reflects the current design philosophy, with heightened emphasis on resilience, presented in the WSDOT *Bridge Design Manual* (BDM) as well as the critical nature of the study corridor.

There are three performance standards for western Washington bridges when evaluated for resilience in a very large earthquake. They are:

Critical Bridges

- Will sustain minimal to moderate damage
- Immediate access for emergency services
- I-5 Study: no structures identified

Recovery Bridges (Lifeline)

- Will sustain some (repairable) damage
- Access shortly after earthquake
- I-5 Study: mainline I-5 bridges, tunnels, and select ramps

Ordinary Bridges (Not Lifeline)

- Meets life-safety requirements (no-collapse)
- Major damage/do service after earthquake
- I-5 Study: all structures not designated Recovery (overcrossings, non-critical tunnels, lid structures not carrying mainline I-5, and ramps, etc.)

Two performance standards were used in this study's evaluation: Recovery and Ordinary.

Recovery bridges serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake.

Ordinary bridges are intended to meet "life-safety" requirements. "Life-safety" requirements allow significant damage after a seismic event. The intent of a "life-safety" threshold is that the bridge does not collapse in a seismic event, preventing loss of life, but allows for significant service disruptions. In some instances, replacement may be required after a large earthquake.

Because tunnels and lid structures can similarly impact I-5, they were also categorized as either Recovery or Ordinary structures.

Significant retaining walls and embankments are those that, if they fail during a seismic event, would block several lanes of I-5, reducing it to one or no lanes in either direction or will block a Recovery access ramp. Several (thirty-eight) of the fifty retaining walls and embankments in the study corridor appear to be Significant.

Geotechnical Considerations

Ground-shaking intensity and geotechnical hazards that can impact the structural performance of bridges, ramps, tunnels, lids, walls, and embankments were considered for this study. Study analysis was based on existing subsurface data and geologic and hazard mapping; no new subsurface explorations were performed. Earthquake-induced geologic hazards that result in a significant reduction in the soils' ability to support the structure, or that result in permanent displacement of the ground on which the structure is founded, pose significant hazards to engineered structures. Using several available resources, classifications for each structure were made including liquefaction, peat-related settlement, cyclic softening, and landslide hazards. In this way, the geotechnical engineer was able to estimate appropriate levels of effort for exploration, testing, and analysis that is critical to performing a vulnerability analysis for the structures in this corridor.

Methodology for Analysis and Level-of-Effort Development

For the purposes of this study, the team established a systematic approach to assess the vulnerability analysis LOE for each structure. Each set of structural as-builts was examined to determine likely seismic vulnerabilities and the analysis measures needed to assess that structure. Additionally, many structural characteristics were captured and added to a database. This information was combined with information provided by WSDOT.

Previous consultant completed seismic projects were used to establish a baseline Seismic Vulnerability Analysis LOE. Factors were then created to scale the LOE based on additional complexities introduced by items such as curvature, multiple frames in-span hinges, deep foundations, multiple foundation types, soil liquefaction, and previous seismic retrofits. All of these elements increase the complexity of the structural modeling and analysis and introduce additional capacity calculations needed for more elements and scenarios (i.e., separate liquefied and non-liquefied analyses). This information was used to develop a systematic approach to estimate the effort necessary to conduct the Seismic Vulnerability Analysis. It was applied to approximately 90 to 95 percent of the bridges in the study.

The remaining study structures are unique and highly complex and required a customized approach to estimate the LOE versus the systematic approach discussed above. The Lake Washington Ship Canal Bridge and its double-deck approach spans, and the lid that supports the Seattle Convention Center (SCC), are examples of unique structures. In general, these highly complex structures will require significant complex modeling and analysis efforts to determine the most probable seismic performance and determine if the structure may be retrofitted to meet the performance criteria assigned to it for this study—or if replacement is a more suitable option.

Geotechnical estimations were developed in a similar manner. A baseline LOE for sampling, testing, and analysis was developed using geotechnical history on these types of projects. Geotechnical hazards were captured for each structure such as liquefaction and landslide hazards and assessed a relative increase in the LOE. Then each structure was scaled comparatively to the baseline structure.

The respective LOE for each bridge/tunnel/lid was categorized as Lower, Medium, or Higher based on the estimated time and effort required to complete the Seismic Vulnerability Analysis. These LOE ranges are as follows:

- Lower (●○○): 225 to 1,150 hours
- Medium (●●○): 1,151 to 4,200 hours
- Higher (●●●): 4,201 to 19,000 hours

The respective LOE for each significant retaining wall or embankment was similarly categorized as Lower, Medium, or Higher based on the estimated time and effort required to complete the Seismic Vulnerability Analysis with generalized ranges represented as follows:

- Lower (●○○): 150 to 550 hours
- Medium (●●○): 551 to 1,150 hours
- Higher (●●●): 1,151 to 2,500 hours

The anticipated deliverable for each Seismic Vulnerability Analysis is a detailed report for each structure, or group of structures, that includes discussion of the criteria applied, the analysis methodologies used, a geotechnical report, and the results of the analysis. Structure descriptions and calculated capacity demand (C/D) ratios for the structural elements would be included, as well as conclusions drawn from the analysis and recommended conceptual retrofit solutions to address vulnerabilities discovered. A rough order of magnitude (ROM) estimate of construction costs associated with the conceptual retrofit measures would also be provided. Finally, detailed calculations and relevant seismic structural model input and output should be provided as appendices.

An example of a recent Seismic Vulnerability Study is included in Appendix E (without geotechnical reports and appendices).

Next Steps

The seismic risk assessment conducted for this report applied performance criteria that exceed "life-safety" to evaluate bridges for resilience in a significant earthquake. This higher standard reflects the current design philosophy, with heightened emphasis on resilience, presented in the WSDOT BDM as well as the critical nature of the study corridor. Applying this standard to all structures results in a significant estimated LOE. Illustrated in the flow chart below (Figure 5) is a recommended set of critical next steps that will provide important information to inform refined cost estimates for the seismic vulnerability phase.

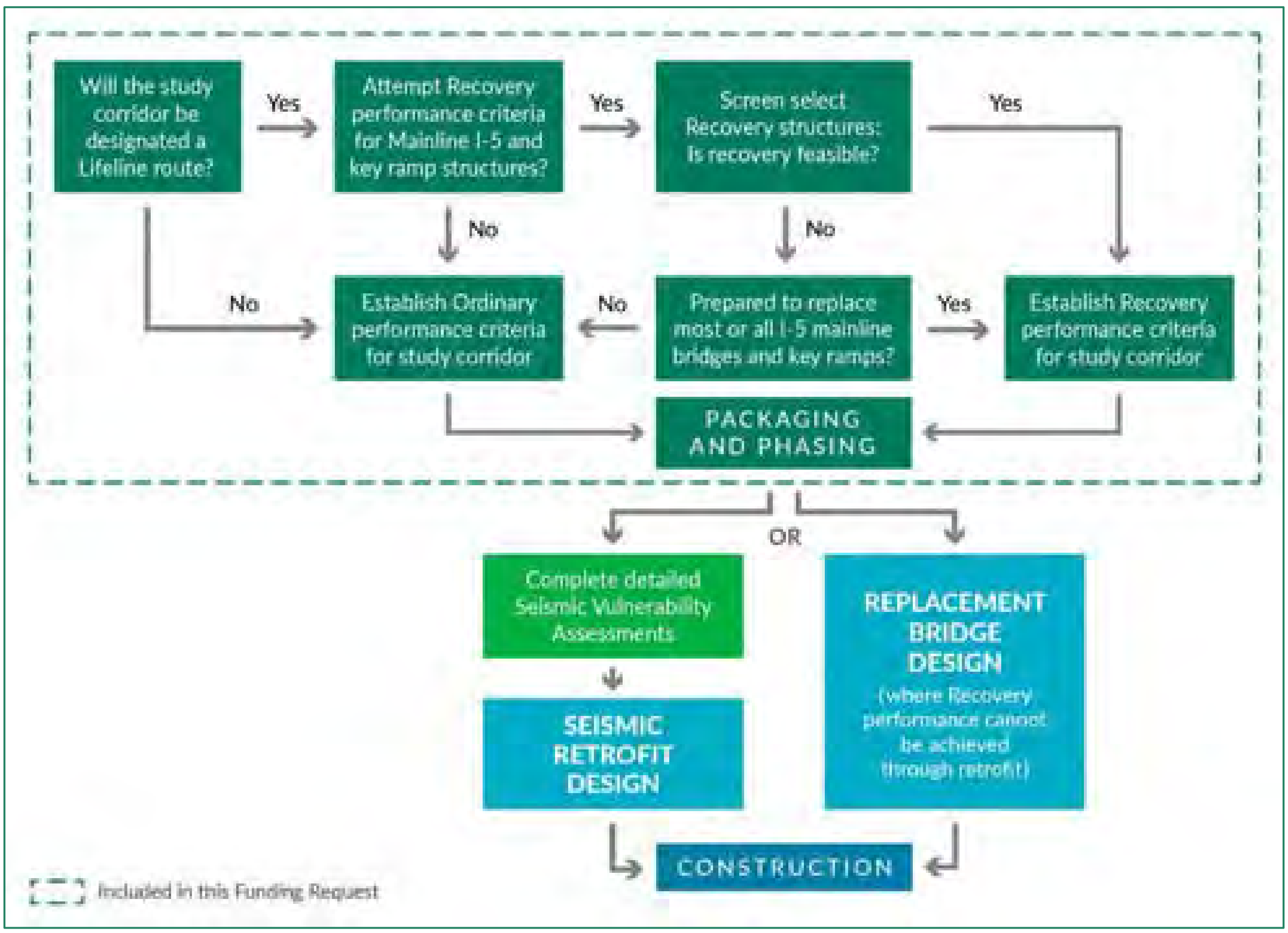


Figure 5: Flow Chart for Next Steps

As a critical first next step, it is important for WSDOT to determine if the study corridor will be designated as part of the Lifeline route. If it is not, lower performance criteria can be applied to all structures, reducing the LOE to conduct the analysis. If a Lifeline designation is adopted, the higher Recovery performance criteria will be attempted, and a screening step will occur to determine if it is feasible to achieve the criteria though retrofit alone. If Recovery performance cannot be achieved through retrofit, replacement would be considered to meet the higher performance level. Based on the outcomes and decisions, WSDOT will recommend packages and phases. Cost estimates to conduct the analysis for the recommended package(s) will then be prepared.

Purpose of the Study

The Interstate (I-5) corridor between the Boeing Access Road and Lake City Way (State Route [SR] 522) is the backbone of the Puget Sound region. Business, industry, residents, and travelers passing through the area rely on this corridor for transport of goods and services as well as themselves. With an average daily traffic (ADT) volume over 200,000 this corridor is Washington State's most heavily used stretch of highway.

In the past, other corridors, namely Interstate 405 (I-405), have been prioritized for seismic retrofit to maximize the state's funding based on "miles upgraded per dollar spent" metrics. Many of the bridges within the study corridor are complex and located in dense urban environments. Because of the corridor's age, complexity in many locations, and challenges with maintenance of traffic (MOT) associated with retrofit construction, the bridges within this corridor have, for the most part, received only minor seismic retrofits.

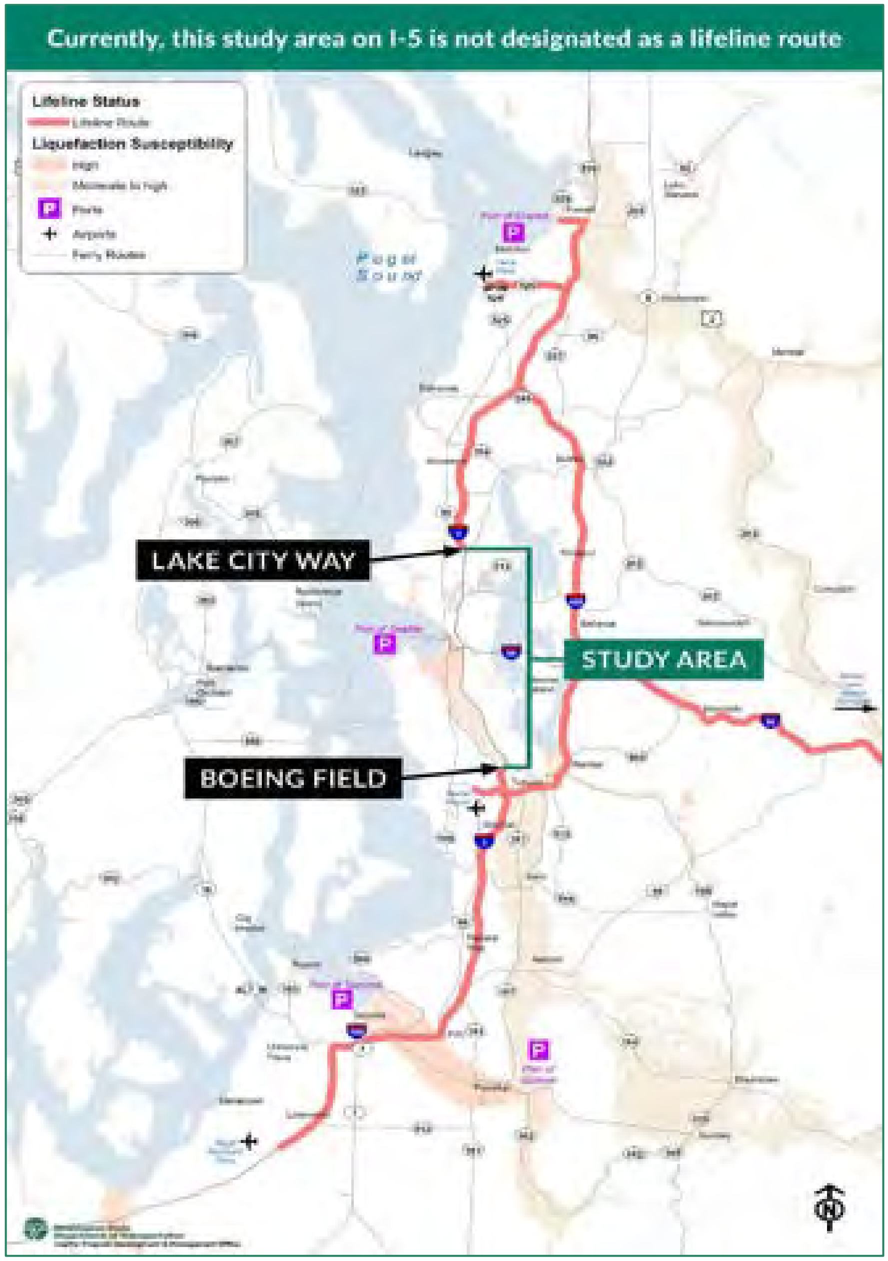


Figure 6: Current WSDOT Lifeline Map Showing Study Limits

The purpose of this planning study is to provide Seismic Vulnerability Analysis recommendations for the structures on the corridor and determine the level of effort (LOE) required. This detailed vulnerability analysis would be the **first of three future phases** to retrofit the corridor to a level indicative of its importance to the state and local communities (see Figure 7).

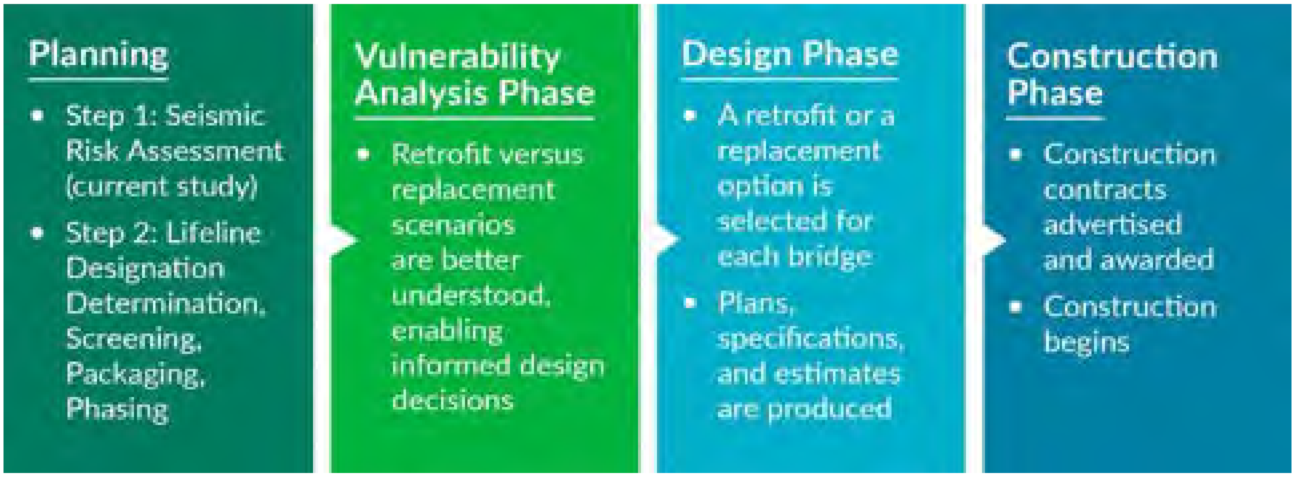


Figure 7: Recommended Program Phases

The Seismic Vulnerability Analysis determines each structure's performance under loading from a large earthquake. The results are compared to established and accepted design criteria, including expected performance. This step is critical as it clarifies both the vulnerabilities and potential retrofit measures that may be implemented to meet the performance criteria. The analysis recommendations included in this study are based on the team's extensive experience with previous seismic analyses of similar structure types and characteristics in the region. Where highly detailed, rigorous analysis is recommended, the team identified an opportunity to potentially reduce seismic retrofit construction costs through such analysis.

The first phase of work would be to perform the recommended Seismic Vulnerability Analyses and determine appropriate retrofit measures to address the identified deficiencies.

In some cases, it may be prohibitively expensive, or even impossible, to achieve the project performance criteria through seismic retrofit. In these cases, a replacement structure would be recommended. Bridge retrofit (or replacement) design funding requirements would be developed in this phase. Rough order of magnitude (ROM) construction costs and other associated funding needs, such as MOT and program management, would also be estimated in this phase.

With the Seismic Vulnerability Analysis complete and a retrofit (or replacement) scheme established, the design phase would begin, building on the Seismic Vulnerability Analysis. It includes development of the contract documents required to advertise each structural retrofit or replacement for construction. The Seismic Vulnerability Analysis computer models and post-processing tools established in Phase 1 may be modified to include seismic retrofits. The structural behavior is then checked to determine if the established seismic criteria can be met. Iterations are completed as necessary, and the results are incorporated into design drawings to be advertised for construction. Detailed construction and other associated project costs would be established in this phase and presented in the Engineer's Estimate.

The third, and final, phase would include the actual construction of the designed seismic retrofits or replacement structures. A much smaller design effort is typically required during construction to answer contractor requests for information and assist with issues that may arise during construction.

Currently, this study funding request does not include the actual detailed analysis, design, construction, or "soft costs." An incredible amount of variability and complex construction impacts are present in this corridor; therefore, each structure needs careful consideration to both assess seismic retrofit needs and reduce the constructability uncertainties. Therefore, it is prudent to develop those costs as more of the program needs are identified in reasonable detail to assign appropriate costs.

By using this approach, a significantly better understanding is obtained in each phase, impacts to travelers and nearby residents can be minimized or mitigated, and the construction cost estimates become far more reliable.

WSDOT has been programmatically working with this process as funding has been made available. This program initially completed seismic upgrades based on now outdated seismic demands and criteria, which had significant demand increases in 2007. Those seismic demands are currently being revised to risk-based values that will be incorporated in the 2023 American Association of State Highway and Transportation Officials (AASHTO) design documents. The seismic retrofit program has continued to evolve and has made excellent progress with aboveground retrofits for the purpose of collapse prevention and life-safety. However, several bridges and stretches of roadway remain where seismic retrofitting is incomplete and minimal retrofitting has been accomplished. The goal of this study is to identify the approach for the Seismic Vulnerability Analysis, LOE required, and next steps to continue this important work and close the "gap" on I-5.

Limits of Study

The study corridor is I-5, between Boeing Access Road and the SR 522 (Lake City Way) interchange (mileposts 158 to 171). As indicated in Figure 8 below, the study limits (north and south end points) connect to WSDOT's currently designated Lifeline route.

The south end of the study corridor includes access to Boeing Field, which is a strategic connectivity point to this corridor as it enables access from the airfield for the movement of goods and services and connects to the remainder of the "Lifeline" route.

Heading north, the corridor parallels Boeing Field toward South Seattle. South Seattle is composed largely of residential areas and heavy industry, including a major Union Pacific Railroad railyard and the Port of Seattle. A major interchange in South Seattle connects I-5 to the West Seattle Bridge via the Spokane Street Viaduct.

Continuing north, past Beacon Hill, I-5 connects to Washington State's largest east–west corridor at Interstate 90 (I-90). This complex interchange network connects I-5 to I-90 as well as to U.S. Coast Guard Base Seattle, Seattle's two largest outdoor stadiums, and the International District.

Farther north, the freeway approaches multiple significant healthcare facilities in or around the First Hill district to the east. The central core of downtown Seattle and access to the waterfront and Colman Dock are to the west. This is also

Figure 8: I-5 Seismic Study Corridor

the southern terminus of the reversible high-occupancy vehicle (HOV) lanes. There are many overcrossings in this area and at the northern limits of downtown. There are also four large lid structures that support Freeway Park and the Seattle Convention Center (SCC).

Once past downtown Seattle, the freeway goes through South Lake Union, home to Amazon headquarters, and travels along the eastern side of Lake Union. I-5 then connects to SR 520, which will soon have new, revised connections to I-5. The freeway then spans across the Lake Washington Ship Canal at Portage Bay on the Lake Washington Ship Canal Bridge, touching down to grade at the University District at the north end. This major bridge is a double-deck structure that carries HOV traffic on the lower deck. The study corridor then continues to the north across Ravenna, ending at the northeastern edge of Green Lake where it meets SR 522 (Lake City Way). From this interchange, the highway continues north through other districts of Seattle, where I-5 is included in WSDOT's designated Lifeline route.

Structures

The WSDOT structures database lists one hundred twenty-three bridge or tunnel structures within the project limits. Additionally, the team identified fifty-three potential retaining walls and "embankment zones" within the study limits. Upon further investigation, the team established thirty retaining walls and eight "embankment zones" as Significant for the purposes of this study. A more detailed breakdown of the structures and the performance criteria applied to them are shown in Figure 9 below. Further information regarding performance criteria is provided in the next section.

Figure 9: Structures and Embankments—by the Numbers

Performance Criteria

WSDOT has identified the following three expected bridge seismic performance classifications for the bridges in Washington:

- Critical bridges
- Recovery bridges
- Ordinary bridges

For the purposes of this study, the team established study-specific criteria for the bridges within the corridor. The intent of the criteria for each bridge was to balance the resilience needs of the Lifeline route and the cost to retrofit these bridges. All bridge structures are designated as either an Ordinary or a Recovery bridge.

Critical bridges are expected to provide immediate access to emergency and similar life-safety facilities after an earthquake. The Critical designation is typically reserved for high-cost projects where WSDOT intends to protect the investment or for projects that would be especially costly to repair if they were damaged during an earthquake. This classification carries the highest seismic resilience. **No bridges** in this study are identified as **Critical** bridges.

Recovery bridges serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake. This classification is neither the highest nor the lowest resilient alternative but rather is the middle alternative for seismic resilience. **Forty-two** bridges (roughly one third) included in this study have been identified as **Recovery** bridges.

Ordinary bridges are those not designated as Critical or Recovery. This is the lowest classification for seismic resilience and is intended to provide for life-safety only. **Eighty-one** bridges (roughly two thirds) included in this study have been identified as **Ordinary** bridges.

Bridge structures were designated as Recovery based on the following criteria:

- All bridges carrying mainline I-5 northbound or southbound traffic
- On/off ramps that are near population centers
- On/off ramps that provide access within a reasonable proximity to maintain connectivity
- Access to important facilities (e.g., emergency medical facilities, Port of Seattle, Colman Dock, etc.)

All other bridge structures are designated as Ordinary for the purposes of this study.

Additional Background

The expected bridge seismic performance classification relates directly to the expected performance of a bridge in a particular seismic ground motion return period. Currently, the WSDOT *Bridge Design Manual* (BDM) considers two seismic ground motion return periods for bridges in western Washington: the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE)—see Figure 10 below. The expected performance for each bridge classification, relative to the seismic ground motion levels considered, is shown in Table 1.

Figure 10: Seismic Ground Motion Definition

The lower-level FEE was not considered as part of this study. In the team's opinion, the FEE criteria are better suited for new construction; therefore, it would be included for the structures in this study only in the event that a replacement bridge is considered the most appropriate resolution to a seismically vulnerable structure. While the FEE could be considered in the Seismic Vulnerability Analysis and resulting retrofit schemes, it may be the controlling criteria in some instances and would likely result in significantly increased retrofit costs that would not necessarily enhance the post-earthquake performance in a design-level event. Caution should be used when determining whether to make retrofit decisions based on the FEE criteria.

Table 1: Reproduction of WSDOT BDM Table 4.1-1

Bridge Operational Importance Category	Seismic Hazard Evaluation Level	Expected Post-Earthquake Damage State	Expected Post-Earthquake Service Level
Ordinary Bridges Western Washington	SEE	Significant	No Service
	FEE	Minimal	Full Service
Recovery Bridges	SEE	Moderate	Limited Service
	FEE	Minimal	Full Service
"Critical Bridges"	SEE	Minimal to Moderate	Limited Service
	FEE	None to Minimal	Full Service

The expected post-earthquake damage states are defined in Section 4.1.3 of the WSDOT BDM as follows:

- **None**: No damage.
- **Minimal**: "Flexural cracks and minor spalling and possible shear cracks." Essentially elastic performance.
- **Moderate**: Extensive cracks and spalling resulting in visible lateral and/or longitudinal reinforcing bars. Bridge repair is likely, but bridge replacement is unlikely.
- **Significant**: "Imminent failure," i.e., onset of compressive failure of core concrete. Bridge replacement is likely. All plastic hinges within the structure have formed with ductility demand values approaching the limits specified in Table 4.1-2 of the WSDOT BDM (Table 2 below).

Additionally, the expected post-earthquake service levels are defined in the WSDOT BDM as follows:

- **No Service**: Bridge is closed for repair or replacement.
- **Limited Service**: Bridge is open for emergency vehicle traffic. A reduced number of lanes for ordinary traffic is available within three months of the earthquake. Vehicle weight restrictions may be imposed until repairs are completed. It is expected that within three months (Recovery bridges) or within three days (Critical bridges) of the earthquake, repair works on a damaged bridge would have reached the stage that would permit ordinary traffic on at least some portion of the bridge.
- **Full Service:** Full access to ordinary traffic is available almost immediately after the earthquake. The expected post-earthquake damage states and service levels of Critical bridges are included in Table 1 above to provide an indication of their expected performance relative to Ordinary bridges. Note that higher ductility values lead to higher amounts of deformation.

Table 2: Reproduction of WSDOT BDM Table 4.1-2

Seismic Critical Member	Displacement Ductility Demand Limits					
	Ordinary Bridges		Recovery Bridges		Critical Bridges	
	SEE	FEE	SEE	FEE	SEE	FEE
Wall Type Pier in Weak Direction	5.0	1.5	2.5	1.5	1.5	1.0
Wall Type Pier in Strong Direction	1.0	1.0	1.0	1.0	1.0	1.0
Single Column Bent	5.0	1.5	2.5	1.5	1.5	1.0
Multiple Column Bent	6.0	2.0	3.5	2.0	1.5	1.0
Pile/Shaft-Column with Plastic Hinge at Top of Column	5.0	2.0	3.5	2.0	1.5	1.0
Pile/Shaft-Column with Plastic Hinge Below Groun	4.0	1.5	2.5	1.5	1.5	1.0
Superstructure	1.0	1.0	1.0	1.0	1.0	1.0

In the context of this study, the agreed-upon intent is to maintain at least one lane of traffic on mainline I-5 for throughput of emergency vehicles and services immediately following an SEE event. This intent is also applicable to select on-ramps and off-ramps, to maintain access on and off of the highway. Bridges identified to meet this intent have been accordingly designated as needing to meet the higher Recovery bridge performance level—which would provide "limited service" after an SEE event. This higher performance level would also help reduce the expected damage state to "moderate" and lead to a faster return to ordinary traffic operations within the corridor.

The remainder of the study bridges are designated as Ordinary bridges. This is traditionally considered "life-safety" performance and the focus of retrofits of these bridges is collapse prevention. Expected damage levels may be significant, and bridges may be unusable and require replacement after an SEE event. By accepting significant damage to these bridge structures, the retrofit construction costs are reduced, accordingly.

WSDOT has historically used a "no-collapse" criteria for seismic retrofit of bridges in its inventory—including those on the existing Lifeline routes. The driving philosophy has been to prevent the loss of life, improve the resilience of the corridor, and maximize the "miles upgraded per dollar spent."

Seismic retrofits have historically focused on those that can be constructed above the existing ground. Some ground disturbance has been required to complete the installation of column jackets, but generally, foundation retrofits have not been implemented. Depending on the soils, detailing, and site constraints, foundation retrofits are generally considered prohibitively expensive where funding is limited and have been deferred to a future phase that would include foundation retrofits when funding is made available.

Study bridges that are designated as Ordinary will generally not be considered as candidates for seismic retrofit of the foundations to minimize construction costs.

Study bridges designated as Recovery will likely need to incorporate some type of foundation retrofit. By including foundation retrofits, service disruptions will be minimized as the resilience of the corridor will be greatly enhanced.

While the focus of this project is the vulnerability assessment of bridges in WSDOT's inventory, three other significant structure types are considered in the corridor. These are lid-type structures, tunnels, and retaining walls and embankments.

Similar to the bridges, tunnels and lid-type structures were designated as Recovery or Ordinary, depending on the potential impact of failure these structures have to mobility within the corridor.

Like bridges, lid structures are named/numbered structures in the WSDOT inventory, and this corridor has four lid structures within the study limits. A similar bridge structure classification of Recovery versus Ordinary was used for these structures. Three of these lid structures support Freeway Park across I-5 north of downtown Seattle and the fourth supports the SCC. The lid supporting the SCC is classified as a tunnel in the WSDOT inventory; however, its seismic analysis will be more similar to a bridge structure than a tunnel. These structures are all designated as Ordinary for the purposes of this study.

Tunnels are also named/numbered structures in the WSDOT inventory, and the study area includes six tunnels within the study limits. As with lid structures, tunnels were categorized using similar bridge structure classification of Recovery or Ordinary. All six of the tunnels in this area are used as on/off ramps and are used for access to/from mainline I-5—two of which have been classified as Recovery for this study, leaving four classified as Ordinary.

Retrofits of tunnel structures are typically quite complicated as there is generally access to the inside of the structure only. Thus, seismic retrofit of tunnel structures may require thickening the tunnel walls or roof on the inside, reducing the usable roadway prism.

Lastly, the potential impact of failing retaining walls and embankments in this corridor was also considered. Retaining walls do not have the same naming/numbering scheme as the bridge structures. However, some very large walls, large multilevel wall systems, and walls are located in highly landslide-prone areas in this corridor. Our team identified walls that appear "Significant," meaning that failure of these walls would have large impacts on mobility on this corridor. Our team also paired the geotechnical assessment of the area's landslide hazards with large walls and/or hillsides and embankments.

It is important to note that the team did not have access to the retaining wall as-built plan drawings, as these can be very difficult to locate in the WSDOT archives. The approximate age of the walls is known based on their location within the corridor. While this does not allow a comprehensive assessment for each wall, it does lend itself to the likelihood that walls in the vicinity of newer bridge structures may have been designed for an SEE event. Most of the significant walls in the corridor were originally constructed in the 1960's and are more likely to show vulnerabilities in an SEE event. Additionally, it is unlikely that most of these walls were designed considering global stability failures because of a seismically induced landslide within the wall limits.

Seismic Vulnerability Analysis Methodologies

To survive a large earthquake, each bridge must have an adequate load path from the superstructure through the substructure and into the foundations. Each element and its connections must be able to resist the deformations and large imposed forces. Each bridge will need to be analyzed using a reliable load path consistent with its most prevalent earthquake-resisting system (ERS). The ERS chosen will consider member characteristics (such as size, reinforcing, and material properties) and boundary conditions. The ERS must provide an uninterrupted load path for transmitting seismically induced forces into the earth, with sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Three primary overarching methodologies are considered and recommended to analyze the bridges within the study corridor. These three methodologies, in order of complexity (least first), are:

- Foundation stability and seat width
- Response spectrum analysis (RSA) and nonlinear pushover
- Nonlinear time history (NLTH)

Foundation Stability and Seat Width

This methodology is recommended for single-span bridges. Single-span bridges have historically performed very well in large seismic events. There are fewer failure modes, which generally leads to good seismic performance. As such, these bridges should not require a three-dimensional (3D) analysis model. However, it is important to determine if the bridges have adequate capacity to maintain stability in a large seismic event. The LOE to perform this analysis is lower than the analysis effort to analyze multi-span bridges.

Analysis for these structures should verify that there is adequate seat width to prevent girder unseating. WSDOT performed many seat width retrofits in this corridor in the early 1990s. Therefore, these single-span bridges are anticipated to have very few, if any, seat width vulnerabilities. However, because of changes in the seismic demands used in current design, this must be verified.

The abutments should be analyzed to determine if these elements present any seismic vulnerabilities. This is particularly true for bridges categorized as Recovery. If the abutments are determined to exhibit significant foundation failures, they should be retrofitted to mitigate the failure and enable a faster return to normal traffic conditions. If the abutments exhibit localized failures, such as shear key failures, the resulting movements may affect the drivability of the bridge after an earthquake. In some extreme scenarios, repair—such as bearing or expansion joint replacement—may need to be performed prior to allowing traffic on the bridge if the failure is not mitigated with retrofit.

RSA and Nonlinear Pushover

Also referenced as the Method D2 procedure of the Federal Highway Administration (FHWA) *Seismic Retrofitting Manual*, this evaluation method is suitable for the vast majority of the study corridor bridges. This analysis uses an idealized elastic response to determine the expected displacement demands. Nonlinear static procedures, commonly referred to as “pushover” analysis, are then applied to determine the practical displacement capacity of each element as it reaches its limit of structural stability.

Seismic displacement demands should be determined from a multimodal RSA using structural analysis software. A sufficient number of modes must be included to account for at least 90 percent participation of the total mass.

Modal response contributions should be combined using the Complete Quadratic Combination (CQC) method. Response of the structure is to be analyzed in two orthogonal horizontal directions and the results combined according to the 100%+30% rule. Vertical acceleration effects typically need not be included in this analysis.

By building these analytical models, the inelasticity of the substructure elements is approximated in the RSA by assuming a reduced moment of inertia in elements expected to exhibit plastic hinging. By generating the nonlinear pushover models from the RSA models, the force redistribution that occurs during deformation is captured. Thus, a more realistic measure of behavior is found than that of an elastic analysis procedure, though some conservatism is built into the degradation and cyclic energy dissipation.

The following assumptions are typically made for all response spectrum models:

- Members are modeled with frame (beam) elements with six degrees of freedom at each joint.
- The superstructure is modeled as a single spline with at least ten equal segments per span. Superstructure frame elements are modeled at the composite neutral axis along the bridge centerline. These elements are then assigned appropriate mass and bending stiffness.
- Interior piers are modeled as a frame with each column connected to a rigid capbeam. Column-to-capbeam joint regions are modeled with rigid links. Appropriate boundary conditions are to be captured. The capbeam stiffness for in-plane bending is made artificially high to compensate for the point load application of superstructure transverse moments.
- Columns are modeled with appropriate fixity at the capbeam and foundation connections in both directions.
- Springs are to be used as appropriate to model appropriate foundation flexibility.
- Columns are modeled with equivalent cracked stiffness. The reduced stiffness values are determined using methods provided in Section 7.3.2.1 of the FHWA *Seismic Retrofitting Manual*.
- Abutment foundation stiffness is to be considered.
- Mass is distributed in accordance with Section 7.3.1 of the FHWA *Seismic Retrofitting Manual*. Live load gravity effects need not be included in the seismic analysis. Additionally, the inertia of live loads is also not included in the analysis.
- Unless otherwise justified, a constant five percent damping coefficient is used for all modes.

Nonlinear Time History

NLTH analysis is generally reserved for irregular complex bridges, or when site-specific ground motions are to be used, as in the case of a bridge of major importance. This type of analysis combines both the demand side of seismic evaluation in the form of earthquake ground motion input and the capacity side in the form of fully cyclic non-linear characterizations.

The analysis can be divided into the following three major steps:

1. An assessment of the seismic ground shaking hazard for the specified ground motion level return period (e.g., SEE ground motion level) at the site is performed and a suite of earthquake ground motions (acceleration or displacement time histories) is produced that are representative of the seismic ground shaking hazard.
2. An analytical model is constructed that includes non-linear material, stiffness, energy dissipation, and compatibility characteristics of the soil, structural elements, and boundary conditions.
3. A dynamic time history analysis is conducted for each ground motion providing displacements and member actions (forces and deformations) as a function of time for a specified earthquake ground motion. These member actions are then post-processed and evaluated to determine the performance of the structure.

Several industry standard software applications are specifically tailored to perform this type of structural dynamic analysis for bridges and transportation structures. Two of the more common of these applications are CSi Bridge and ADINA, and one of these would likely be used where NLTH analysis is needed along the I-5 corridor.

The NLTH analysis method is particularly useful for structures that have irregular geometry or large variations in mass and stiffness properties. It can provide both numerical and visual insight into structural motions and performance and is essential when refined seismic motions and explicit energy dissipation is considered. This method can be used as a primary analysis tool or as a refinement in an effort to avoid potentially costly retrofits.

While this analysis method can be one of the most accurate means of assessing the seismic performance of a bridge structure, it requires considerable computational effort. Additionally, a significant level of skill is needed in interpreting the results. Preliminary solutions from simpler methods should always be obtained before undertaking an NLTH solution, to bound the results and check for meaningful results. It should be noted that once the model is completely constructed and all design parameters are complete, it typically takes many hours and sometimes days just for the analysis to run. It is a relatively common practice to use a powerful computer that is dedicated to completing this analysis—even with today's modern computers. The cost of this analysis method can be many times that of other methods and should be carefully weighed against the potential benefits before it is selected.

Geotechnical Considerations

As part of this study, several geotechnical hazards throughout the corridor were identified. These hazards can exacerbate the seismic ground shaking, impose permanent ground deformations on the structures, and lead to worsened structural performance. Often several of these conditions require additional analysis effort to understand the seismic behavior of a bridge.

A geographic information system (GIS)-based screening process was applied to identify structures susceptible to the following geohazards:

- Peat-related settlement
- Cyclic softening
- Liquefaction
- Mass-wasting (landslides)

For this screening-level evaluation, structure geometry, represented by line data in GIS, was overlayed on published geologic and geohazard maps. Structures that intersected entirely or in part with map areas associated with a specific geohazard were identified as susceptible to that geohazard using binary (yes/no) classification. For the case of landslides, a three-tiered susceptibility classification was created, indicating relatively higher and lower susceptibility based on a combination of existing landslide inventory and susceptibility maps.

Peat-related Settlement

Areas of likely peat accumulations were extracted from the Geologic Map of Northeastern Seattle (Booth et al. 2009[1]) and the Geologic Map of Seattle (Troost et al. 2005[2]), including zones mapped as peat (geologic map unit Qp), wetland deposits (Qw), and lake deposits (Ql).

[1] Booth, D., Troost, K., Schimel, S. (2009). Geologic map of northeastern Seattle (part of the Seattle North 7.5' × 15' quadrangle), King County, Washington. U.S. Geological Survey Scientific Investigations Map SIM-3065. Scale: 1:12,000.

[2] Troost, K., Booth, D., Wisher, A., Schimel, S. (2005). The geologic map of Seattle – A progress report. U.S. Geological Survey Open-File Report OF-2005-1252. Scale: 1:24,000.

Cyclic Softening

Cyclic softening is temporary strength loss due to seismically generated excess pore pressure in fine-grained soils. Areas potentially prone to cyclic softening were extracted from the geologic maps of Booth et al. (2009) and Troost et al. (2005), including Holocene-age lake deposits (geologic map unit Ql) and recessional glaciolacustrine deposits (Qvrl).

Liquefaction

Areas prone to liquefaction were extracted from the City of Seattle's Potential Liquefaction Area online layer (Seattle GeoData 2012a[3]), as defined in the City's municipal code for Environmentally Critical Areas (ECA), Section 25.09.012.A.2.

Landslides

Structures were assigned tiered susceptibility classes to deep-seated landslides based on a combination of inventory and susceptibility maps. "Level 1" classification (higher relative susceptibility) was assigned to structures that crossed existing deep-seated landslide deposits mapped with light detection and ranging (LiDAR) by the Washington Geological Survey (Mickelson et al. 2019[4]) or Schulz (2005[5]). Structures that fell within a 200-foot radius of historical deep-seated landslides recorded by the City of Seattle (Seattle GeoData 2012b[6]) were also assigned a "Level 1" classification. Locations of historical landslides in the City inventory are mapped as points that are alternately associated with the address of the property closest to the landslide, the address of the property affected by the landslide, or the address of the person(s) reporting the incident. A 200-foot buffer was assigned around each point because of the spatial uncertainty of the landslide locations.

"Level 2" classification (lower relative susceptibility) was assigned to structures that crossed mass-wasting deposits identified in geologic maps (map unit Qmw; Booth et al. 2009 and Troost et al. 2005), potential landslide areas defined by City of Seattle Municipal Code Section 25.09.012.A.3.b (Seattle GeoData 2012c[7]), or within 40 feet of existing deep-seated landslide deposits mapped by Mickelson et al. (2019) or Schulz (2005).

Seismic Site Class

Each structure was assigned a seismic site class based on site class maps produced by the Washington State Department of Natural Resources (DNR 2010[8]). Where a structure crossed more than one site class zone, the lowest site class was assigned. If a structure crossed a peat or cyclic softening zone, the site class was downgraded to classes E–F and D–E, respectively.

Roadway Sections Susceptible to Landslides

Roadway sections susceptible to landslide undermining, displacement, or debris over the roadway were mapped in GIS based on intersection with the landslide inventory and susceptibility maps listed above.

---

[3] Seattle GeoData. (2012a). Potential liquefaction area: Liquefaction zones ECA. Available at: https://data-seattlecitygis.opendata.arcgis.com/datasets/SeattleCityGIS::liquefaction-zones-eca/about. Accessed: August 4, 2022.

[4] Mickelson, K., Jacobacci, K., Contreras, T., Gallin, W., Slaughter, S. (2019). Landslide inventory of western King County, Washington. Washington State Department of Natural Resources, Washington Geological Survey Report of Investigations 41.

[5] Schulz, W. (2005). Landslide susceptibility estimated from mapping using light detection and ranging (LIDAR) imagery and historical landslide records, Seattle, Washington. U.S. Geological Survey Open-File Report 2005-1405.

[6] Seattle GeoData. (2012b). Historic landslide locations ECA. Available online at: https://data-seattlecitygis.opendata.arcgis.com/datasets/SeattleCityGIS::historic-landslide-locations-eca/about. Accessed: August 4, 2022.

[7] Seattle GeoData. (2012c). Potential landslide areas. Available at: https://data-seattlecitygis.opendata.arcgis.com/datasets/SeattleCityGIS::potential-landslide-areas/about. Accessed: August 4, 2022.

[8] DNR. (2010). Seismic Ground Response. Available at: https://www.dnr.wa.gov/geologyportal. Accessed: August 4, 2022.

Additionally, roadway sections immediately downslope of the City of Seattle potential landslide areas and recorded shallow landslides were considered susceptible to landslide debris running out over the roadway. In some cases, susceptible roadway sections were extended beyond their intersection with mapped landslide deposits due to upslope topography that was visually similar to known landslide areas in LiDAR-derived slope- and hillshade-map (NV5 Geospatial 2021[9]).

AASHTO Design Ground Motions

With regard to the AASHTO design ground motions, they have changed significantly over time in this corridor. For example, the "Design" peak ground accelerations (PGAs) have increased approximately tenfold for rock in Seattle since seismic design of bridges in 1961. PGAs that take into consideration site-specific soil effects (i.e., Site Class) are even larger than those shown in Figure 11 below:

- **1949 and older versions:** No explicit seismic design provisions; effectively PGA = 0.
- **1961:** Based on a ~PGA of 0.02 to 0.06 (depending on foundation type). EQ demand = CD where C is earthquake acceleration (i.e., PGA) and D is the deadload of the structure.
- **1973:** Same as 1961.
- **1992:** Acceleration coefficient based on PGA from 500-year ground motion. For Seattle, the acceleration coefficient (PGA) for rock conditions increased to 0.33. Entire response spectrum based off of this single acceleration coefficient.
- **1996:** Same as 1992.
- **1998 LRFD:** Same as 1992.
- **2002:** Same as 1992.
- **2006 LRFD:** Same as 1992.
- **2007 LRFD:** Changed to 1,000-year ground motion; ground motions from 2002 United States Geological Survey (USGS) National Seismic Hazard Mapping Project. Design spectrum not exclusively based on PGA. However, rock PGA increased to ~0.44g.
- **Current:** Rock PGA 0.45g.

9 NV5 Geospatial. (2021). USGS 3DEP King County, Washington Delivery 1 Lidar. NV5 Geospatial Technical Data Report. Resolution: 1.5 ft. Available at: https://lidarportal.dnr.wa.gov/. Accessed: August 4, 2022.

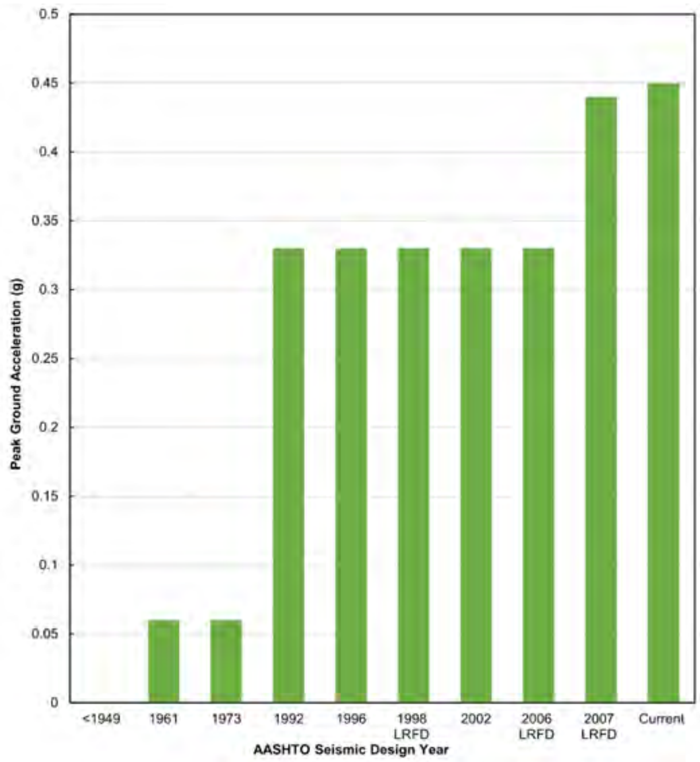


Figure 11: Peak Ground Acceleration (on Rock) Changes over Time

Level-of-Effort Estimation

As previously noted, there are typically two phases to preparing a set of construction plans for advertisement. This study is focused on the first phase—a Seismic Vulnerability Analysis to determine the deficiencies. Once the vulnerabilities are determined, a plan can be formulated to address the deficiencies.

The Seismic Vulnerability Analysis LOE estimation is based largely on data from more than one hundred consultant completed seismic retrofit projects in the Puget Sound region. It also accounts for efficiencies gained from established proprietary tools that have been generated to develop the seismic models and post-process the large amount of data to compute C/D ratios for each element.

For the purposes of this study and to achieve the schedule, the team established a systematic approach to assess the Seismic Vulnerability Analysis LOE for each structure. Each bridge is categorized into one of the following four classifications:

- Single-span structures
- Typical structures
- Complex structures
- Unique structures

The respective LOE for each bridge/tunnel/lid structure was categorized as Lower, Medium, or Higher based on the estimated time and effort required to complete the Seismic Vulnerability Analysis for these structure types. These generalized ranges are represented as follows:

- Lower (): 225 to 1,150 hours
- Medium (): 1,151 to 4,200 hours
- Higher (): 4,201 to 19,000 hours

The respective LOE for each significant retaining wall or embankment was similarly categorized as Lower, Medium, or Higher based on the estimated time and effort required to complete the Seismic Vulnerability Analysis with generalized ranges represented as follows:

- Lower (): 150 to 550 hours
- Medium (): 551 to 1,150 hours
- Higher (): 1,151 to 2,500 hours

Single-Span Structures

As the name suggests, the bridges have a single span and will be the most straightforward to analyze. This bridge type is most commonly a concrete superstructure (precast/prestressed concrete girder, concrete slab, concrete T-beam, or concrete box girder bridge) supported by a concrete abutment—often founded on piles.

Typical Structures

Most of the bridges within this corridor were designed and constructed in a similar time frame (early 1960s); therefore, many of the details are similar when similar site characteristics are present. Most of the bridges in this corridor are classified as a Typical structure for the purposes of this study. These bridges can consist of any superstructure type, be supported on single-column or multi-column bents, and be founded on deep or shallow foundations. These bridges are generally repetitive in nature; but can have some complexity and/or variability in their details.

Complex Structures

Several of the structures in this corridor have been classified as Complex structures. These bridges are similar in some ways to those classified as Typical structures; however, experience building seismic models and post-processing the large volumes of data in the analysis has shown that the analysis LOE grows with complexity. A higher baseline LOE is assumed for Complex bridge structures.

Unique Structures

Structures considered to be "Unique" were deemed too complex to systematically estimate the Seismic Vulnerability Analysis and design effort. Relatively few of these structures are located within the study limits and include structures such as the Lake Washington Ship Canal Bridge, the Freeway Park lid structures, and the tunnel structures. It is highly recommended that each structure identified as Unique create a basis-of-design document prior to initiating the Seismic Vulnerability Analysis. This will help to clarify the input parameters for the design team when future phases are implemented.

Methodology for LOE Estimation

The team used existing databases containing large amounts of information for the corridor bridges, including the BEISt database, which includes the as-built drawings and photos and a large internal WSDOT database created and populated in coordination with universities for other purposes.

The Quickbase online software application is used for this study and is highly customizable. Each bridge within the study corridor has an entry in the database. The information in the internal WSDOT database was imported into Quickbase and many additional fields were created to capture and reflect pertinent information used to establish the LOE to complete the Seismic Vulnerability Analysis. These additional fields included items such as the degree of curvature, the presence of in-span hinges, and landslide hazards. Quickbase also supports the use of mapping software. Each bridge was tagged with a geographic coordinate to identify its location on a map. These locations are linked to the database entries for each bridge.

Each structure was reviewed in detail and a large number of characteristics were captured in Quickbase. In some instances, this required verification of the WSDOT information (e.g., number of spans) and in other instances it required establishing additional criteria (e.g., degree of curvature). As this information is populated and examined, the story of the level of complexity for each bridge unfolds.

Much of this information is critical for determining the LOE to analyze a bridge and is a good starting point to form analysis needs and even potential retrofit scenarios.

The baseline bridge is a three- to five-span single-frame structure supported on piers comprising single or multiple columns. This bridge is on a tangent or nearly tangent alignment and founded on spread footings. The geometry is relatively consistent, and skews are minimal. For the baseline bridge, it is imagined that a spline model is constructed using structural analysis software, such as CSi Bridge. The "spline" is effectively a single element along the superstructure's center of gravity and piers are also constructed using frame elements connected at the appropriate center of gravity. The seismic analysis performed using this spline model will use an RSA and a pushover analysis to determine the displacement demands and capacities. The component capacities will be determined using software such as Excel, Mathcad, and Xtract. Additionally, a small/medium length Seismic Vulnerability Report is assumed to accompany an appendix of calculations and select computer output. The report and appendices should be required for each structure to summarize the large amount of data that are developed as part of the analyses.

With consultant partner assistance, WSDOT has completed a large number of seismic vulnerability analyses, many of which included similarly aged WSDOT bridges located in western Washington. The recorded hours required to complete these projects were examined thoroughly and used to establish an analysis LOE for the baseline "basic" and baseline "complex" bridge. **The baseline "basic" bridge analysis LOE is estimated at about 230 hours** to complete a Seismic Vulnerability Analysis and report. Complex bridges inherently require significantly more effort than a basic bridge and are still subject to the same additional LOE multipliers that are applied to the basic bridges. Our research of previously completed projects indicates that a suitable **LOE of the baseline "complex" bridge analysis is estimated at 450 hours** to complete a Seismic Vulnerability Analysis and report. Neither baseline LOE includes contingency.

Only "unique" structures that are not in close proximity to other structures should be analyzed as a standalone project. Bridges in this corridor should be packaged into bundles based on common characteristics to the maximum extent possible. The reason for this is twofold. First, efficiency is gained as processes are established by design teams. Some local consultants have established a robust process for developing models, establishing capacities, compiling and post-processing the data, and completing quality assurance and quality control (QA/QC) of the work. Still, it may take team members time to familiarize themselves with the tools if they have had long breaks in between the work. Second, many of these bridges are located within complex interchanges or in close proximity to other bridge structures. This interaction cannot be ignored and each bridge in close proximity should be analyzed by the same team to determine the possible interaction and associated ramifications. It should be noted that the LOE of each structure indicated in this study assumes that it is part of a packaged bundle, for the most part.

The LOE to complete a seismic retrofit analysis increases with complexity. Using this baseline, and the additional characteristics captured within Quickbase, other "typical" bridges were then estimated based on their level of complexity. In some instances, additional effort is required to build out the additional details into the seismic models. In others, additional effort is required to post-process significantly more data resulting from multiple iterations to capture variable soil parameters, for instance.

It should be noted that the amount of data generated by these seismic vulnerability analyses is incredibly large. Frequently the final report includes only adequate input to reconstruct the model, and selected output showing critical demands in order to make the reports less cumbersome to accept and use going forward.

The factors that were used to increase the LOE estimate for each bridge include:

- Additional spans
- Curved
- Skewed
- Multi-frame
- Grillage/shell models required
- Multi-pile deep foundations
- Shaft deep foundations
- Multiple superstructure sections
- Box girder superstructure sections
- NLTH
- Previously widened
- Previously retrofit
- Multilevel bridge
- Liquefiable soils
- Adjacent structure interaction
- Landslide hazard

The factors affecting the LOE are somewhat variable from one structure to the next and some caution is advised when considering multiple factors concurrently. However, the factors included in this study were agreed upon by the team's WSDOT Bridge and Structures representatives and have been proofed against several seismic retrofit projects and determined to be in reasonable agreement for somewhat complex bridge structures in the Puget Sound region.

A description of the factors applied to the LOE follows.

Additional spans require that additional geometry be incorporated into the seismic model. The LOE typically increases with the size and length of the bridge. There are several ways to categorize this, but for the purposes of this study, the number of spans was used to determine an appropriate LOE. This geometry can sometimes vary based on specific details for each bridge. Therefore, it is not only a matter of incorporating more elements. Modeling the additional geometry accurately, including column heights and sizes, span configurations changes, crossbeams, and differing boundary conditions all require additional effort to incorporate correctly. More importantly, the amount of data to post-process increases with each span. However, there is typically a limit. As the lengths of the bridges in this study increase, they also typically become more repetitive. If the details do not vary significantly, then a comprehensive model for the entire bridge may be limited to five frames (typically around 15–20 spans). Therefore, a variable factor is applied to the LOE for the additional spans as indicated, below:

- Bridges comprising **6–11 spans: a 1.3 factor applied**
- Bridges comprising **12–17 spans: a 1.6 factor applied**
- Bridges comprising **more than 17 spans: a 2.0 factor applied**

Curved bridges are typically those that have 20 degrees or more of sweep angle between abutments. This requires generating the complex seismic model geometry in greater detail, and accounting for the rotated local axes of each element in the model. **A 1.1 factor is applied** to capture this effort in the Seismic Vulnerability Analysis.

Skewed bridges are typically those that have piers that are skewed to the superstructure by 30 degrees or more. Skew effects have major effects on the performance of a bridge and its elements in a seismic event. These impacts are typically accounted for in the Seismic Vulnerability Analysis. **A 1.1 factor is applied**.

Multi-frame bridges require constructing multiple models: a tension model that assumes the frames are independent of one another and free to move longitudinally, and a compression model that assumes that the joints have closed and the frames will move in unison. The effort to build the two models combined with the LOE to post-process significantly more data is quite large. **A 1.25 factor is applied**.

Grillage/shell models are both challenging to construct and require a high LOE to post-process the data. These models are different from a spline model in that they use longitudinal members that represent the primary beam elements and transverse elements that represent the deck slab and diaphragm elements in what is known as a “grillage.” The significant increase in the number of elements comes with a larger amount of data to post-process. This level of refinement is warranted when the structural complexity cannot be accurately defined with a simple spline model. **A 1.75 factor is applied**.

Multi-pile deep foundations include bridge foundations placed on a group of steel or concrete piles because of poor soil conditions. These piles can be driven or drilled as a means of installation. These multi-pile foundations can be modeled using different techniques, but in all instances, a series of iterations is necessary to capture the expected behavior. The modeling techniques use a series of elastic springs placed below ground and the interaction of the soil and the deep piles needs to be refined through iteration. If these springs are too stiff, then they attract an unrealistically large amount of loading, and if they are too soft, then they attract too little load. It is typically necessary to use two software applications (e.g., CSi Bridge and LPile or Group) to perform the iterations of the spring stiffness values to an appropriate representation of the soil-structure interaction (SSI)—resulting in an accurate seismic response for the design earthquake. **A 1.25 factor is applied**.

Shaft deep foundations are instances where a bridge substructure is founded on large-diameter concrete drilled shafts because of poor soil conditions. Installation of these drilled shafts uses an auger (or similar mechanism—these are never driven) to reach a required elevation below grade. Modeling of drilled shaft foundations uses a similar process to multi-pile foundations described above; however, less effort is generally required for one drilled shaft, as opposed to an array of piles. Therefore, the iterations are typically faster, and the amount of post-processing is reduced, relative to multi-pile foundations. **A 1.1 factor is applied.**

Multiple superstructure sections also require more effort to complete the analysis. Different superstructure sections result in different stiffnesses, centers of gravity, and mass along the bridge length. Building these definitions can be particularly complicated when the as-builts are either short on information or difficult to decipher (generally because of illegible scans). When the superstructure has changes to its cross section along its length, these changes need to be quantified to build an accurate representation of the bridge in the structural modeling software. Additional post-processing effort is also necessary as the capacity will most certainly change along the length as well. **A 1.1 factor is applied**.

Box girder superstructures are a common bridge structure within this corridor. These can be designed using mild reinforcement for shorter spans (most common in this corridor) and post-tensioned to span longer distances. Additionally, they add significant torsional stiffness, which makes them an excellent bridge choice for curved bridge alignments. However, they require additional time to capture the geometric changes, and the many reinforcement changes that are typical along the bridge length. They are typically integral at the intermediate piers and require additional analysis to make sure that they are not vulnerable along the length because of the overstrength moment and shear that results because of the column seismic hinging. **A 1.1 factor is applied**.

Nonlinear time history analysis (described in the previous section of this report) is reserved for special scenarios where a refined level of analysis is used to better understand the bridge seismic behavior. This takes a considerable amount of time, but results in the most realistic assessment of the structure's behavior and reduces the conservatism associated with the more simplified analytical procedures. This methodology is assumed applicable only to Recovery bridges and recommended only where the team thinks that the additional analysis effort may offset the potential cost of significant retrofit or replacement. **A 2.5 factor is applied**.

Previously widened bridges were originally constructed (typically in the early to mid-1960s) and then later widened to accommodate additional superstructure width for additional traffic lanes. This poses challenges to both the seismic analysis and the post-processing. A second set of bridge drawings must be thoroughly examined, and the added elements must be incorporated into the seismic model. These added details frequently result in additional effort to make sure that the section properties, stiffness, and boundary conditions are appropriate. Frequently, the detailing at the old-to-new structure connection interface was not adequately designed for a seismic load case, which can require additional effort to establish appropriate retrofit concepts to be implemented in the design phase. **A 1.2 factor is applied**.

Previously retrofitted bridges are those that were initially constructed (typically in the 1960s) and later seismically retrofitted in an attempt to improve seismic performance. Unfortunately, the retrofit strategies completed in this corridor are typically not comprehensive and frequently were completed to a lower seismic standard with an initial focus on preventing the most glaring deficiencies to provide maximum reward with low investment. Verification of the previous retrofit adequacy is required, and additional time is required to examine the retrofit plan drawings and incorporate these modifications. This results in additional effort to build the seismic model and to perform additional post-processing calculations to verify their adequacy. **A 1.1 factor is applied**.

Multilevel bridge structures are those that have integral superstructures at different elevations at the same pier location. They are present at several locations in this corridor—typically at large multilevel, multi-ramp interchanges and reversible HOV locations. The complex details used to connect the columns, crossbeams, and superstructures require a large effort to develop detailed seismic models that capture the seismic behavior. Additionally, there are many more elements to be checked, and many more calculations developed to validate the load path of the system. Determining appropriate retrofit solutions is complicated by the additional deck level and the resulting connection to the columns that affect the seismic response significantly relative to a single-level structure. **A 2.0 factor is applied**.

Liquefiable soils are geotechnical seismic hazards that impact the structural performance of the bridge. Effectively, the ground shaking can decrease the soil capacity, which in turn impacts the boundary conditions for the bridge in the seismic event. As there is no surety if or when the supporting soil will liquefy, two seismic models must be developed: the first representing liquefied soils and the second representing non-liquefied conditions. These two models act as "bookends" to capture the probable range of behaviors of the structure and require additional effort to iterate the SSI and to post-process all of these data. **A 1.4 factor is applied**.

Adjacent structure interaction: Most of the bridges within the study corridor are isolated structures and, consequently, the seismic performance and deformations encountered are based purely on the characteristics of the bridge and the earthquake it is subjected to. However, several bridges are adjacent to another existing structure. When these bridges experience seismic ground shaking, they come into contact with the adjacent structure—thereby altering their response. This alternate behavior must be captured and adds complexity to the Seismic Vulnerability Analysis and must be considered with any proposed retrofit measures. **A 1.2 factor is applied**.

Landslide hazard: Several large hillsides located within this corridor have a significant landslide hazard present. This is another geotechnical hazard that has a significant impact on the seismic response of the bridge structure when it is founded on a hillside prone to a landslide under a seismic event. This requires significant coordination with the geotechnical engineer to capture the appropriate soil behavior and then requires significant effort to develop the additional load cases and boundary conditions. These are critical to understanding the seismic performance for the vulnerability analysis and establishing appropriate retrofit solutions. **A 1.2 factor is applied**.

Geotechnical Analysis LOE Estimates

The LOE of a geotechnical seismic study depends on several factors including:

- Type of geologic hazards that may be present at a site
- Availability/unavailability of existing subsurface explorations and laboratory testing
- Type and size of engineered structure (e.g., single-span vs. multi-span bridge vs. retaining wall)
- Geotechnical parameters needed for structural analyses and design (e.g., response spectra vs. spectrum-compatible earthquake time histories for NLTH analyses)

Consequently, the effort for the geotechnical seismic analyses can vary widely for a given structure type (e.g., short single span vs. large multi-span bridges) and among different types of structures (e.g., bridges vs. walls vs. tunnels vs. engineered roadway embankments). Consequently, the team developed cost estimating formulas for bridges, walls/embankments, and tunnels that factor in the size of the structure, the type(s) of geologic hazards present at a particular structure location, and the types of analyses needed to address those hazards and provide seismic ground motions for analyses of the structure.

The approach for developing the LOE for these factors was to develop a "base" effort for the simplest structures where no geologic hazards are present, and then add additional effort based on the size of the structure and the presence and type of geologic hazards at the structure location. The following describes the base efforts for the various structures and how the additional effort for structure size and geologic hazard are typically captured.

Base Level of Effort

Common to a geotechnical seismic assessment for any structure is the collection and synthesis of existing subsurface data, and development of a subsurface soil/geologic model on which subsequent geologic hazards are assessed and geotechnical analyses are performed.

- For the simplest bridges (i.e., single-span), subsurface conditions are assessed at each abutment (total of two locations).
- For relatively short (i.e., approximate 200 to 400 feet or less in length), walls and embankments on competent soils or rock, a single subsurface/geologic model is often developed at the “critical” wall/embankment location, e.g., where the structure is the tallest or has the most adverse geometry.

For simple bridges (e.g., single-span) or short (approximately 200 to 400 feet or less) walls and embankments founded on relatively competent rock (typically Seismic Site Class B) or very dense/hard glacially overridden soils (typically Seismic Site Class C) or other soils that are medium dense or denser (e.g., weathered glacially overridden soil or glacial outwash deposits, typically Seismic Site Class D), often the existing subsurface information developed for the original design of the structure is sufficient for development of a subsurface soil/geologic model and that no new explorations or laboratory testing is needed. This base condition also assumes that it can be determined by simple inspection of the existing subsurface information that potential seismic geologic hazards do not present a performance or design issue at the structure location. This base LOE is shown in Table 3 below.

Additional Subsurface Explorations

Where geologic hazards are identified at a given structure, it is assumed that the existing subsurface information will need to be augmented with additional subsurface explorations and laboratory testing. The team also assumes that because the explorations are to support seismic evaluations, the subsurface exploration will include geophysical measurement of dynamic soil stiffness (i.e., dynamic compression and shear wave velocity measurements). The cost for a single subsurface exploration, including geophysics, geotechnical laboratory testing on selected samples retrieved from the exploration, drilling subcontractor costs, and labor for field coordination and logging, are provided on Table 3. The subcontractor drilling and laboratory testing costs were converted to an LOE engineering “hours” on Table 3 by assuming typical burdened geotechnical engineering hourly rates.

Geologic Hazard/Site Response Analyses

Where geologic hazards are present at a structure, additional geotechnical engineering effort is required to assess the hazard and develop mitigation options. In addition, for very large or critical structures, (e.g., the Lake Washington Ship Canal Bridge) time history analyses will be required for an NLTH analysis of the structure. The geotechnical engineer must develop multiple spectrum-compatible earthquake time histories as input for the NLTH analyses. The effort for each of these geotechnical analyses is provided on Table 3.

Bridge Size

Where a geologic hazard is present, the number of potential new subsurface explorations and the effort for a given geologic hazard needs to increase proportionally by the number of locations where the bridge and the ground interact, i.e., the number of bridge foundations or bents. A span factor is used to account for the number of bridge spans along with the geotechnical analysis effort for the various geologic hazard/site response analyses for a given bridge location. The formula by which the geotechnical engineering effort in the number of bents and geologic hazard/site response analyses are provided in Table 3.

Table 3: Geotechnical LOE Estimating Formulas

Geotechnical Task		Hours
Base	Bridges	85
	Tunnels	85
	Walls/embankments	40
Exploration (typical boring, ~50' to 70')	Drilling subcontractor	35
	Geophysics (Vs/Vp)	20
	Laboratory	9
	Coordination/observation	40
	Total per boring	104
Additional hazard/site response analyses (where hazard is present)	Peats/cyclic softening (1D site response)	80
	Liquefaction	80
	Level 2 landslide (pseudo-static)	60
	Level 1 landslide (pseudo-static plus 2D seismic deformation)	200
	Ground motions for NLTH	240

LOE Estimating Formulae

Bridge geotechnical engineering Level of Effort = (base + ∑[hazard analyses]) × (span factor) + (boring) × (#spans) / (span factor) × (hazard analyses flag)

where:

Span factor = 1+(#spans-1)/10

Hazard analyses flag = 0 (no hazards), 0.5 (peats/cyclic softening, liquefaction, Level 2 landslide), 1 (Level 1 landslide, NLTH)

Wall/embankment geotechnical engineering Level of Effort = (base + ∑[hazard analyses]) × (length factor) + (boring) × (length (ft) / 200 ft) × (hazard analyses flag)

where:

Length factor: if length ≤200', 1; otherwise = 1 + length (ft) / 400 ft

Hazard analyses flag = 0 (no hazards), 0.5 (peats/cyclic softening, liquefaction, Level 2 landslide), 1 (Level 1 landslide, NLTH)

Tunnel geotechnical engineering Level of Effort = base + 1D site response +1 boring (tunnels not in hazard areas)

Wall/Embankment Size

For walls and embankments, the length of the wall or embankment and the impact of a geologic hazard that may be present on the global stability have more of an impact on the LOE for seismic analyses than the height. Consequently, similar to bridges, the number of potential new subsurface explorations and the effort for a given geologic hazard needs to increase proportionally by the length of the wall or embankment where global stability is being impacted. The formula by which the geotechnical engineering effort factors in the wall/embankment length and geologic hazard/site response analyses is provided in Table 3.

Unique Structures

Lid-type Structures (Lids)

Four lids are located within the I-5 corridor extents. Three of these lids support the City of Seattle's Freeway Park and the fourth supports the SCC.

Freeway Park is supported by three distinct lid structures. The first is identified as bridge 5/548PW, supports the “Box Gardens” area of the park, and is located on the southwest side of the park. This structure spans across the southbound off-ramp to 6th Avenue. The second is bridge 5/548PS and supports the area of the park between the Seneca Street off-ramp and the southern side of Seneca Street. This structure spans across NB I-5 supported on a viaduct structure (above the express lanes), southbound I-5, and the southbound off-ramp to 6th Avenue. The third lid is bridge 5/548PN and supports “Seneca Plaza,” located on the north side of Seneca Street. This structure spans across similar I-5 elements as 5/548PS. As these structures are not carrying mainline I-5 structures, all three of the Freeway Park lid structures have been classified as Ordinary bridges.

The structure of each of these lids has been subdivided into separate “units.” Each unit uses precast/prestressed girder superstructures with a cast-in-place (CIP) concrete deck. Many slopes and vertical steps are located along the top surface of these structures. As such, in many instances, each unit acts as a standalone structure. The diaphragms are also irregular in these structures and span between the girders to support the bridge deck. The girders are typically supported on steel pintles on one end and steel rollers at the other end for a pinned/roller support scheme. These bearing systems are in turn supported on concrete capbeams atop pier walls founded on spread footings or counterfort-style abutments.

The geometry for all three Freeway Park lids is highly irregular. Many of the girders have been splayed to accommodate the geometry and in other locations many of these girders have been cantilevered. Longitudinal expansion joints have been added between units. The pier walls are not aligned, which adds to the geometric irregularities in these structures.

The east and west abutments for these structures have complex detailing. Existing counterfort gravity walls were previously reconfigured and had the top portion of the wall removed, a new capbeam constructed on the vertical wall stem, and a new augercast concrete pile was cored through the existing wall footing.

A significant amount of consideration will be required to assess the adjacent structure interactions. The Spring Street Bridge, Seneca Street Ramp Bridge, Seneca Street Bridge, and several planter boxes are within very close proximity to these lid structures. Additionally, the footings for many of these columns are located within 1 inch of adjacent structure foundations. This is because the existing bridges were built approximately 10 years prior to the creation of Freeway Park. There may be instances where removal of planters should be considered to reduce the potential for adjacent structure interactions and resulting vulnerabilities.

The SCC is the fourth lid structure included in the study limits. Similar to the Freeway Park lid structures, it was added after the original I-5 construction, does not carry mainline I-5 traffic, and has been classified as an Ordinary structure. Although technically classified as a tunnel in the BEISt database, the seismic analysis will be more in line with a lid structure than a tunnel. Therefore, it is considered a lid for the purpose of this study. This multi-span structure is highly irregular and the 8th Avenue Bridge bisects the building. The structure reuses the original CIP walls and is supported by columns placed between the northbound and southbound lanes of I-5. It also has complex building framing that composes the roof of the bridge (and the floor of the building) to span across mainline I-5. Understanding the seismic behavior of this structure will require modeling the building and the lid that it is built on together. An NLTH grillage model should be used to analyze this structure and the impacts of the adjacent structures will need to be considered.

Analysis of these structures should be completed using a grillage model with shell elements for the deck structure. This level of modeling complexity is required to account for the splayed girders, high skews, highly variable diaphragms, and various deck conditions.

Capacity protection for some of these elements may be very difficult to achieve. The pier walls are heavily reinforced, which is likely a result of designing for lateral loading using an equivalent lateral force. It is anticipated that a rocking analysis will be required to assess the stability of the substructure in a seismic event. It may be determined that the structure rocking is initiated prior to yielding, thereby dissipating energy. The performance criteria must be clarified in the basis of design, but the analysis may determine that capacity protection is not required in the foundation to satisfy the "no collapse" criteria associated with Ordinary bridges.

These structures were originally analyzed for a much smaller seismic force and using a methodology that is inconsistent with today's techniques. The large mass of the materials on the bridge deck (mainly concrete planters, sidewalks, and soil) will generate large movements and forces. The deck itself is not planar, so there will be large thrust forces in the deck. The girders will almost certainly require additional girder stops at the supports to reduce out-of-plane stresses. The concrete pier walls are likely deficient longitudinally and have an aspect ratio approaching 1:1 in some locations; therefore, overturning will need to be assessed and eventually prevented. The foundations are in such close proximity to other foundations that substantial retrofits may be required and will need to consider the demands from multiple adjacent bridges. Additionally, the bridge foundations in other locations may require retrofit if it is determined that rocking is excessive, resulting in seismic instability. Seismic isolation bearings may be required to reduce the seismic forces imposed on the substructure in an upper-level event, which would add further complexity to the structural model(s).

Finally, some of the existing as-built plans for these structures are illegible. To properly model these elements and determine their capacities, detailed site investigation, measurements, and in situ non-destructive testing will be necessary. This in situ testing may require significant traffic control to provide a safe work environment. These potential needs have been considered in the lid-type structures where the phenomenon is present.

Lake Washington Ship Canal Bridge

The Lake Washington Ship Canal Bridge is a two-level deck bridge with a total length of about 4,400 feet and designated as a Recovery bridge. The upper deck carries ten lanes of traffic, and the lower deck carries four lanes. The main span consists of multiple simple- and continuous-span steel trusses with a total length of approximately 2,300 feet.

Figure 12 shows the main span elevation, Figure 13 shows the south approach elevation, Figure 14 shows the north approach elevation, Figure 15 shows a typical cross section of the north approach, and Figure 16 shows a typical section of the concrete slab at the beginning of the south approach.

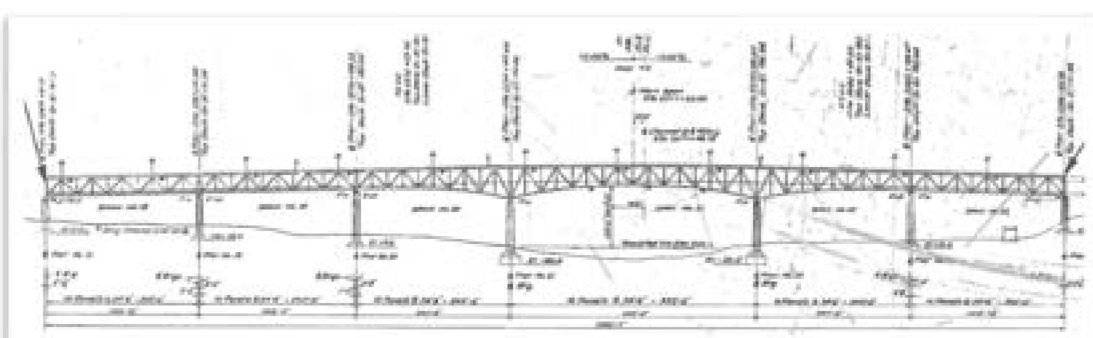


Figure 12: Main Span Elevation

The north and south approach span superstructures consist mainly of concrete multi-cell box girders with integral cross beam-to-column connections (see section below). In the original configuration, some columns are split in the middle to allow for longitudinal thermal movement.

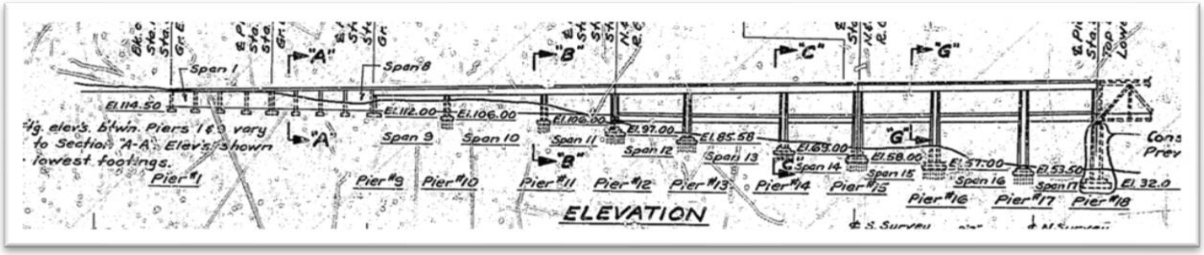


Figure 13: South Approach Elevation

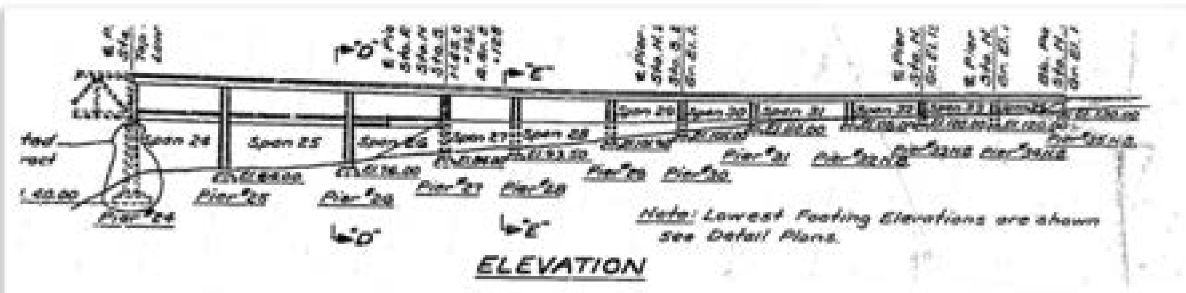


Figure 14: North Approach Elevation

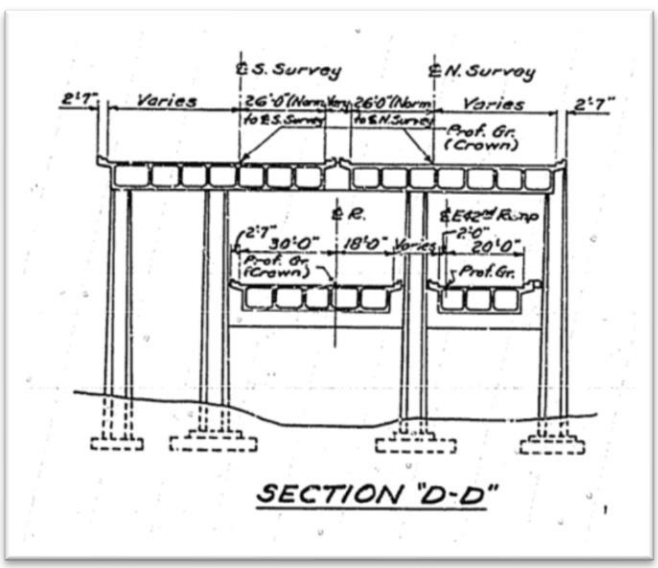


Figure 15: Typical Span Cross Section

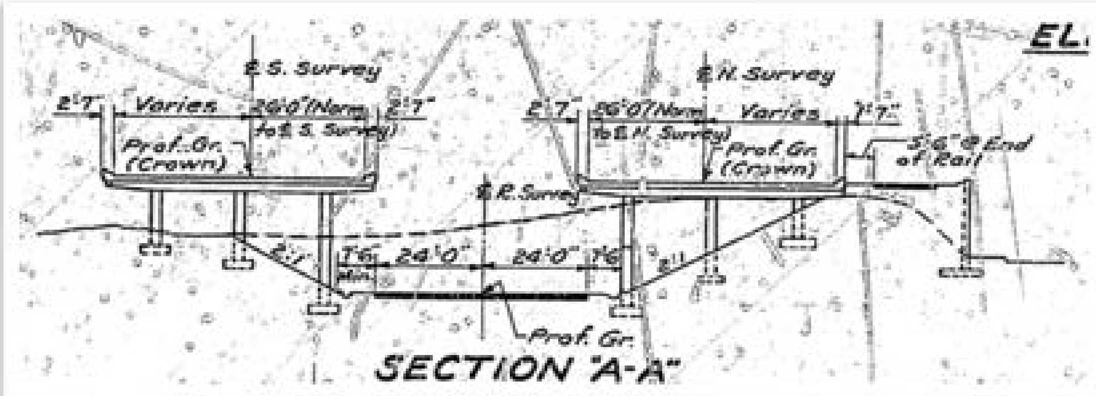


Figure 16: Typical Concrete Slab at Beginning of South Approach

There are thirty-five piers included in the entire bridge and all but piers 10 through 17 are founded on spread footings. Piers 10 through 17 use pile caps founded on an array of concrete piles because of poor soil conditions under the south approach.

The bridge was previously retrofitted in two stages. The Stage 1 retrofit addressed the main steel truss spans and included elements such as longitudinal restrainers, bumper blocks, and bearing collars. No major substructure retrofits were completed on the main span piers. The Stage 2 retrofit focused on the concrete approach spans. These seismic retrofits included steel column jacketing of cruciform columns, replacing top of column/superstructure connections with sliding bearings, strengthening upper- and lower-level cross beams by use of post-tensioning, and adding catcher blocks under some of the lower-level crossbeams.

To date, this structure does not yet have a comprehensive set of seismic retrofits to resist an upper-level seismic event. WSDOT completed an extensive analysis in 2014 that used a linear time history analysis methodology. The previous analysis considered Ordinary performance and did not use an NLTH because of the data limitations and long runtimes. This analysis found that some elements previously retrofitted were seismically vulnerable and that many of the substructure elements that had not been previously retrofitted were deficient (notably crossbeams, columns, and footings). It recommended that three priorities for work elements be completed: Priority 1, collapse prevention retrofit work; Priority 2, strengthening seismically deficient elements; and Priority 3, foundation retrofits and soil remediation for landslide mitigation.

Much of this work could be built upon in a future analysis—it would be particularly helpful to define the criteria. WSDOT could perform a high-level assessment to determine the relative impacts of using Recovery performance criteria and better determine the practicality of meeting reduced ductility requirements. Moreover, it is recommended that this be completed before initiating a future analysis.

Assuming that Recovery performance criteria can be met, it is recommended that a Seismic Vulnerability Analysis be performed on this bridge using an NLTH approach. This would result in the highest level of accuracy and the least amount of conservatism in the analysis. This NLTH analysis may be able to be completed using CSi Bridge, but a software application such as ADINA may be more efficient to complete the analysis.

To develop an approximate LOE estimate to complete an NLTH analysis on this bridge, our team leaned heavily on recently completed NLTH analyses completed on other structures on the West Coast. For this estimation, it was assumed that CSi Bridge will be used; however, alternative software (e.g., ADINA) could be used with a similar LOE. Our team initially estimated the hours to complete the modeling effort using a baseline LOE per linear foot of bridge. This was then revised to reflect the anticipated analysis and modeling requirements specific to the Lake Washington Ship Canal Bridge, based on specific details shown in the drawings and site characteristics. These details were notably the number of required ground motion time histories, site-specific potential geotechnical hazards (including liquefaction anticipated between piers 10–17 of the south approach), the number of potential moment-curvature/hinge elements, the number of concrete pile foundations, the number of different superstructure types in the bridge, built-up steel truss detailing, multilevel superstructures, architectural treatments of existing columns, and existing seismic retrofit measures. Additionally, existing retaining walls associated with this bridge will need to be considered.

Hollow-Core Pile Bridges

Bridges in this study considered "hollow-core pile bridges," which are those structures that were constructed with hollow prestressed/precast concrete piles that extend, continuously, from the pile tip to the crossbeam—essentially acting as both a pile and a column. These piles were first driven to a specified tip elevation, and then extend to the bottom of the bridge capbeam in each bent. The bridge capbeams were then cast in place and girders were set on top of the capbeams to support the bridge deck. The capbeams are connected to the hollow-core pile-columns by means of a reinforced concrete plug (typically 1–2 column diameters in length) that forms a solid section. Eight study bridges include this substructure type. Notably, there are several additional instances where this foundation type was used initially. However, the column and pile were completely filled with concrete during construction and, therefore, are not considered hollow-core pile bridges for the purposes of this study.

This bridge type has thus far been minimally retrofitted, only adding longitudinal restrainers to mitigate a seat width deficiency. WSDOT has been working with several universities to determine appropriate retrofitting techniques for bridges founded on hollow-core piles. Older studies found that there may be a risk that the columns may crush, or "implode," toward the column interior, potentially leading to an abrupt loss of strength, resulting in collapse with little or no warning.

The most recent study, completed in 2020 by the University of Washington, asserted that a more likely failure mechanism was transverse cracking occurring at the end of the CIP plug resulting in prestressed strand debonding in the column. This study further asserted that steel column jackets may be sufficient to prevent this failure mechanism.

WSDOT's Bridge Design Office is initiating an analysis of four hollow-core pile-supported bridges, two of which are within the study corridor (5/578E and 5/578W). Once available, the results of this analysis will be used to inform the Seismic Vulnerability Analysis of other structures within the study corridor supported on hollow-core piles.

The hollow-core pile-supported bridges in the study corridor should be screened to determine if Recovery performance criteria can be achieved. Highly refined structural analysis that may significantly reduce the conservatism of simpler modeling methods may show that retrofits meeting Recovery performance criteria are feasible if the displacement demands are reduced sufficiently or seismic isolation methodologies are implemented. For the purposes of this study, it is assumed that a detailed Seismic Vulnerability Analysis is performed, though it may confirm that no feasible method of retrofit sufficiently addresses identified vulnerabilities to a Recovery performance level.

Tunnels

The project includes six CIP reinforced concrete cut-and-cover tunnels. All but one tunnel are single-cell, three-sided or four-sided, rigid frame structures. There is one two-cell tunnel with a bottom slab and a series of columns comprising the middle wall. The two-cell tunnel has bridge columns that pass through the intermediate wall section. One tunnel supports a ventilation building and another tunnel supports bridge columns that are founded on top of its roof.

Seismic analysis of tunnel structures is proposed to be performed by either pseudo-static or dynamic SSI. Two types of seismic loading are traditionally applied to tunnels: ground shaking and ground failure. For rectangular tunnels the ground shaking effects need to be evaluated for racking, axial, and bending deformations. Ground failure effects need to be evaluated for fault rupture, tectonic uplift/subsidence, liquefaction, settlement, lateral spreading, slope instability, and increases to lateral earth pressures. At the six tunnel locations in the corridor, the ground failure effects need not be considered.

Historically, tunnels have typically performed better than aboveground structures during earthquakes due in large part to reduced amplification of ground motions. Shallow cut-and-cover type tunnels are more vulnerable than deep tunnels because of higher excitation of ground motions near the surface. Because tunnels are immersed in and constrained by the geologic medium, they are affected by the adjacent ground deformations/strains as opposed to acceleration values. Tunnel sections in competent rock, or in stiff soils at significant depths, are of lower concern as the shear deformations tend to be quite low. By contrast, tunnel sections in shallow and soft soils are more vulnerable.

The relative stiffness of the tunnel structure to the surrounding soil medium, as well as the surrounding soil properties, affect the viable analytical methods. If the structure is perfectly rigid, then no distortion due to racking will occur. If the structure stiffness is approximately equal to the surrounding soil, then the racking distortion is the same as the soil deformation. If the structure is flexible relative to the surrounding soil, then the racking distortion will be greater than the soil deformation. The comparative effects are similar for the longitudinal force effects and to some extent for ground failure effects.

Seismic analysis of tunnel structures is based primarily on the ground deformation as opposed to an inertial force approach. Evaluation procedures typically use a simplified analytical method or a more complex numerical modeling approach. Simplified analytical methods include evaluating the shear deformation of the structure based on the free-field deformations of the surrounding soil. This approach is typically conservative, particularly in soft soils. If the structure is not in a uniform soil stratum, additional structures are involved (bridge foundations, ventilation building, etc.), or if the shear deformations are large enough to cause inelastic deformation of the structure, then a more refined modeling method is recommended—such as inelastic SSI analysis. Refined modeling can be in the form of pseudo-static, pseudo-dynamic time history, or dynamic time history analysis, in increasing LOEs.

The vulnerability analysis for all six tunnel structures in this corridor should be in accordance with the requirements and recommendations of the following publications:

- Seismic Retrofitting Manual for Highway Structures: Part 2 – Retaining Structures, Slopes, Tunnels, Culverts, and Roadway (FHWA-HRT-05-067)
- AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, First Edition, 2017
- WSDOT BDM
- Technical Manual for Design and Construction of Road Tunnels – Civil Elements, 2009 (FHWA-NHI-10-034)

To estimate the detailed Seismic Vulnerability Analysis LOE for the tunnels in this corridor, a similar approach to the typical bridge LOE estimation was used. These tunnels are similar to typical bridges in that they are relatively uniform in their construction and the LOE to perform the analysis will use similar procedures. For the purposes of this study, a “baseline” tunnel is one that is in a relatively uniform soil stratum, has two or fewer distinct structural sections, has no significant structure interaction with other structures such as bridge or building foundations or ventilation buildings, and is analyzed for Ordinary performance criteria. This tunnel would use a simplified analytical method for evaluating soil shear deformations and structural models to evaluate strain limits based on the displacements.

The baseline tunnel's vulnerability assessment is estimated at approximately 360 hours.

Tunnels evaluated for Recovery performance criteria will require more rigorous SSI modeling and analysis, as will Ordinary tunnel segments that were constructed in complex or multiple soil strata, involve other adjacent structures, or experience inelastic shear deformations.

SSI modeling and analysis, using software such as Plaxis or FLAC, will add approximately 60 percent more effort for the baseline Seismic Vulnerability Analysis. Each additional unique segment of tunnel will add approximately 30 percent more effort to complete the SSI modeling and analysis. Additional structure modeling and analysis will add approximately 25 percent more effort for each additional segment to the Seismic Vulnerability Analysis.

Significant Retaining Walls

As previously discussed, a large number of retaining wall structures are located within this corridor. To our knowledge, no seismic retrofits have been made to any of the retaining walls within this corridor. Retaining wall failures resulting from high seismic events likely do not pose as high a risk to loss of life as those of bridges. However, failure of these walls can result in collateral damage to the highway system and significantly impact the resilience of the corridor. Therefore, the corridor was assessed using tiered criteria to identify potentially significant wall locations:

1. **Geometrically:** Tall walls and/or large multilevel wall systems whose failures would likely result in large impacts on the mobility of the corridor were identified as "Significant."
2. **Geotechnically:** Walls identified as geometrically "Significant" were screened for proximity to geotechnical hazards (e.g., landslide-prone areas) that may impact the global stability of the walls (refer to the Methodology section of this report for more information).

There are multiple wall types within the corridor, and without having as-built plans, each wall type is not necessarily known. However, based on our knowledge of the corridor, the team anticipates that most of these walls consist of the following types: CIP cantilever walls, CIP counterfort gravity walls, secant/tangent pile, and secant/tangent pile with tie-backs. It is expected that some of these shallow retaining wall types have been founded on deep foundations. In each case, as-built information will need to be collected prior to performing an analysis.

Analysis of each wall will need to include checking both the structural integrity of the wall under seismic loading and global stability of the wall. Structural integrity is important because even if the wall does not fail, it may sustain sufficient damage that it is not safe to pass adjacent to (or behind, depending on the configuration). Global stability failures could result in either (1) undermining the roadway, removing multiple lanes, or (2) a large embankment or hillside spilling down onto the roadway, blocking multiple lanes.

Proposed Next Steps

The seismic risk assessment conducted for this report applied performance criteria that exceed "life-safety" to evaluate bridges for resilience in a significant earthquake. This higher standard reflects the current design philosophy, with heightened emphasis on resilience, presented in the WSDOT BDM as well as the critical nature of the study corridor. Applying this standard to all structures results in a significant estimated LOE. Illustrated in the flow chart in Figure 17 below is a recommended set of critical next steps that will provide important information to inform refined cost estimates for the seismic vulnerability phase and lays out the proposed process to progress the seismic retrofit strategy of this corridor.

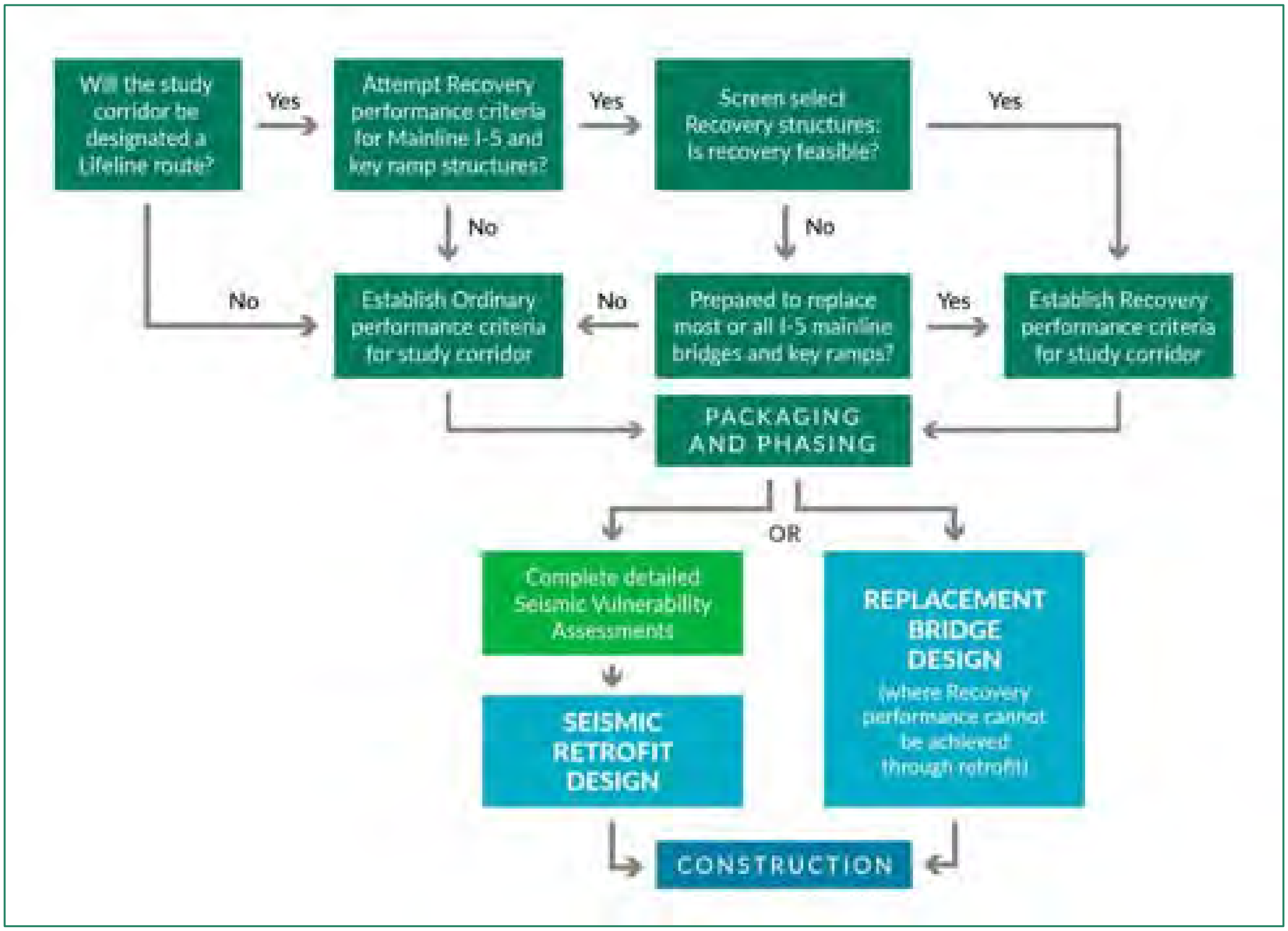


Figure 17: Next Steps Flow Chart

As a first next step, it is important for WSDOT to determine if the study corridor will be designated as part of the Lifeline route. If it is not, lower performance criteria can be applied to all structures, lowering the LOE to conduct the analysis. If a Lifeline designation is adopted, the higher Recovery performance criteria will be attempted, and a screening step will occur to determine if it is feasible though retrofit alone. If Recovery performance cannot be achieved through retrofit, replacement would be considered to meet the higher performance level. Based on the outcomes and decisions, the team will recommend packages and phases. Cost estimates to conduct the analysis for the recommended package(s) will then be prepared.