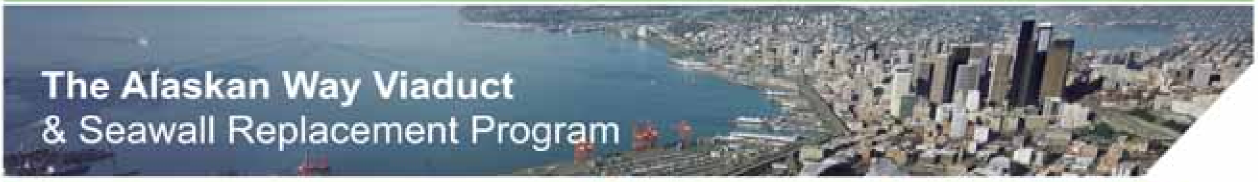

Seismic Vulnerability Analysis Report

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SR 99: Alaskan Way Viaduct & Seawall Replacement Program

Seismic Vulnerability Analysis Report

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The SR 99: Alaskan Way Viaduct & Seawall Replacement Project is a joint effort between the Federal Highway Administration (FHWA), the Washington State Department of Transportation (WSDOT), and the City of Seattle. To conduct this project, WSDOT contracted with:

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Executive Summary

The risk of an earthquake causing the Alaskan Way Viaduct to fall down is significantly higher than was previously thought. Until now, it was estimated that it would take seismic ground motions with a 210-year return period to initiate collapse of the Viaduct. In practical terms, this translates to an approximate 1-in-20 chance in the next ten years of an earthquake sufficient to cause portions of the Viaduct to collapse. We found that an earthquake capable of initiating collapse of the Viaduct has a much shorter expected return period of 108 years. This translates to approximately a 1-in-10 chance in the next ten years of an earthquake that would cause portions of the Viaduct to collapse, or roughly double the previously identified risk. This change in risk is based on new geotechnical information and a better understanding of local and regional seismic behavior.

This review was prompted by new data on the frequency and distribution of earthquakes in the region. To better understand how the Viaduct would react during an expected seismic event, we used advanced structural analysis techniques to reexamine the interaction of local soil conditions and the Viaduct structure that could cause its collapse.

The evaluation of the Viaduct's structural capacity largely verified previous studies. Therefore, the higher collapse potential for the Alaskan Way Viaduct is primarily based on the expectation of increased seismic demands on the structure.

1.0 General Discussion

1.1 Background

The Alaskan Way Viaduct is a 2.1-mile long, circa-1950, reinforced concrete, double-level roadway structure built along the City of Seattle's waterfront (see Figure 1.1). The Alaskan Way Viaduct was designed and built in two phases. The first phase, which includes the elevated structure north of Railroad Way, was completed in 1952 and was designed by the Seattle Engineering Department (SED). The second phase was completed in 1956 and was designed by the Washington State Department of Transportation (WSDOT). This report presents that analysis of Bent 83 which is in the portion of the Viaduct designed by SED. See Figure 1.2 and 1.3.

The seismic vulnerability of double-deck highway facilities such as the Alaskan Way Viaduct started to become a focus of attention after the 1989 Loma Prieta earthquake in California collapsed long segments of the Cypress viaduct structure. The Cypress viaduct was a double-level reinforced concrete structure with some similarities to the Alaskan Way Viaduct. On closer inspection, the two highway facilities are significantly different based on their respective structural systems and detailing of reinforcement.

In 1995, the University of Washington (UW) conducted a number of studies to identify the seismic vulnerabilities of the Alaskan Way Viaduct for WSDOT. One study included a structural analysis of the SED structures (Knaebel et al. 1995). Another study included a similar analysis of the WSDOT structures (Ryter et al. 1995). These studies were more focused on identifying the structural deficiencies in the Viaduct than determining what level of earthquake could collapse the structure. The UW studies used the seismic demands from code prescribed and site-specific ground motions with 500-year return periods.

After the 28 February 2001 Nisqually earthquake damaged the Alaskan Way Viaduct sufficiently to require temporary closure of the facility, WSDOT commissioned the Structural Sufficiency Review Committee (SSRC) to quickly evaluate the Viaduct seismic vulnerability. As part of the SSRC report, an estimate of the return period for a seismic ground motion that could cause loads in excess of the structure stability limits and initiate collapse of the structure was presented. Based on information available at the time, SSRC estimated that a ground motion with a 210-year return period was sufficient to compromise portions of the Viaduct.

At approximately the same time, it was recognized that the adjacent City of Seattle Alaskan Way Seawall was also vulnerable to collapse from two sources: 1) deterioration of the Seawall structures due to time and damage from marine organisms and 2) earthquake induced structural collapse. Because of joint concerns regarding the interrelated potentials for seismic collapse of the Alaskan Way Viaduct and the Alaskan Way Seawall, the Alaskan Way Viaduct and Seawall Replacement Project (AWVSRP or Project) evolved. AWVSRP has since conducted a number of studies regarding the seismic vulnerability of the Viaduct and Seawall, including their existing conditions. Both the Seawall and Viaduct are in deteriorated states given their age. As such, they are more vulnerable to earthquakes now than when constructed.

A ground motion 'return period' expresses the annual rate at which a ground motion level is exceeded at a site. It is a convenient way to express the percent probability of ground shaking occurring or being exceeded for any period. Return periods do not imply that the ground motion occurs once every certain number of years at a site. However, it can be used to predict the probability of reaching that level of shaking in any period. Earthquake return periods coupled with specialized knowledge of regional and local seismicity, combined with thorough structural analysis enables the prediction of a structure's stability. This prediction should not be regarded as a precise answer. There are many underlying assumptions used to determine the seismic ground motion and calculate how that motion affects the structure. Each assumption has a range of uncertainty. These uncertainties can overlap and accumulate. Consequently, a given return period should not be viewed as a precise point but as a bounded range of values in which a given event is likely to occur.

1.2 Seismic Vulnerability

Three major sources of a seismically induced collapse of the existing Alaskan Way Viaduct are explored in this analysis: 1) failure of the structural frames (above ground), 2) failure of the foundations (below ground), and 3) failure caused by mass movement of liquefied soil surrounding the Viaduct foundations following failure of the adjacent Seawall. These components are interrelated, time dependent, and should not be treated independently. Any evaluation made to estimate a seismically induced failure of the Alaskan Way Viaduct should address all three of these components.

Seismic effects on structures are a function of the structures mass, stiffness, strength, ductility, as well as subsurface conditions and ground motions. In a seismic event, the mass of the structure largely does not change. The stiffness of the structure can change rapidly with time during the seismic event as the structure becomes damaged as a result of structural components exceeding their elastic limits. In general, a decrease in structural stiffness during the seismic event softens the response and limits further increase of seismic force demands on the structure. These favorable conditions however require special ductile detailing of key components within the seismic load path, similar to what current design standards require. Ductile detailing of key components was not common practice during the era that the Alaskan Way Viaduct was designed and built. With proper ductile detailing and controlled localized damage, the seismic response of a structure can increase energy dissipation and further dampen the seismic force the structure needs to resist. However, as the damage increases, the probability of collapse also increases. The 1950 vintage Viaduct does not have modern reinforcement detailing for ductile seismic behavior. The Viaduct has little reserve capacity if damaged during an earthquake. The Viaduct has also deteriorated over time and withstood a similar size earthquake in 1965. As a result, in its current deteriorated condition, the Viaduct is more vulnerable to seismic events than when new.

The foundation stiffness during seismic ground motion is affected by the soil stiffness, seismic movements, and the degree to which the soil liquefies. For the soils that provide lateral support for the Viaduct, the foundation stiffness is a function of the lateral load. In general, as the lateral load increases, the stiffness of the foundation decreases. The soils along the waterfront are loose fills that often overlay loose or soft natural marine deposits. With these subsurface conditions and the right ground motions, the soils can liquefy and effectively lose most of their ability to resist lateral movement. As the soil liquefies, there is a significant reduction in the stiffness of the soil-structure system. There is also a significant reduction in the foundation's capacity to resist seismic lateral load. At the same time, liquefaction can reduce the vertical capacity of the foundation. As the liquefied ground settles, it can drag the supporting piles with it; effectively increasing the vertical load on the foundation piles.

The presence of the Seawall can also affects the stability of the Viaduct's foundations. The Seawall confines the soils around the foundations. Should the Seawall fail, it can initiate mass movement of the soil around the foundations. This soil movement would decrease the capacity of the foundations by removing the supporting soil and impose large lateral loads on the foundations. If the soil does not liquefy, the Seawall may withstand an earthquake of sufficient magnitude to initiate collapse of the Viaduct. This assumes the Seawall is in it original conditions; not its current deteriorated state. If a significant proportion of the soils do liquefy, the Seawall is likely to fail and the resulting mass movement of the soil alone may collapse the Viaduct. Estimation of the ground motion that leads to soil liquefaction is integral in predicting the survivability of the Viaduct.

Soil liquefaction and associated effects on structures may not occur simultaneously with the peak earthquake ground motions. Observations during the Nisqually earthquake noted that the onset of liquefaction occurred several minutes after severe ground movement had stopped. An assessment of the return period for earthquake ground motions that may affect stability of the Alaskan Way Viaduct also needs to consider the affect that ground motion may have relative to liquefaction induced instability of the Alaskan Way Seawall.

1.3 Analysis Procedure

A representative Viaduct bent was selected to evaluate systematically the frame, foundation, and soil conditions relative to the latest seismic ground motion data. The analysis worked as an independent verification of previous UW and SSRC findings relative to structural capacities. The SSRC structural capacities were also compared to the new seismic ground motion information as part of the validation to this study.

Bent 83, south side of Madison Street, was analyzed in the transverse direction by non-linear static procedures for several foundation conditions. The selected bent is an interior bent of a typical three span, four bent structure. Foundation conditions including fixed base, non-liquefied soil conditions using non-linear lateral springs, and a liquefied soil condition also using non-linear lateral springs were considered. Bent 83 was chosen because it is representative of a large number of other bents along the waterfront and because its location puts it in a potentially liquefiable soil zone with the Seawall present.

Non-linear static analysis procedures were used to determine where the structure is expected to form a collapse mechanism. The procedure systematically tracked the structural period of vibration for primary lateral modes, equivalent lateral acceleration levels corresponding to the capacity of specific components, component ductility, and structural damping. Appendix A gives a more detailed discussion of the non-linear static procedures. These results were then compared relative to the site-specific Expected Earthquake (EE) response spectra (Shannon & Wilson, Inc. 2004). The Expected Earthquake is the 108-year ground motion (50 percent probability of exceedance in 75 years). Local non-linear pushover analyses of representative frames were carried out to develop the capacity curves for the given bent (see Appendix B). The capacity curves were then converted to an equivalent single-degree-of-freedom (SDOF) system. Two non-linear static procedures using the equivalent SDOF system were then followed to determine the performance of the structure relative to Expected Earthquake response spectra.

Two of the three major sources of seismic concern for the Alaskan Way Viaduct stability were tracked with this non-linear static analysis procedure. These are deficiencies in the frames (above ground) and deficiencies relative to the foundation (below ground). The third source of seismic vulnerability for the Alaskan Way Viaduct stability, which is mass movement of soil accompanying a Seawall collapse, is a separate but related issue.

The circa-1950 reinforced concrete structural frames have several sources of deficiencies because the arrangement of reinforcement is not sufficient to provide ductile seismic behavior. These include lack of confinement for the concrete, limited shear reinforcement in the members and joints, low embedment length for the main reinforcement, and bar splices that may not develop full bar capacities. Each deficiency was addressed in the analysis regarding its effects on the structure's ability to withstand seismic loads and displacements. Appendix C describes the procedures to account for bar development and splice issues. Appendix D discusses member and joint shear capacity procedures.

The pile-supported foundations, in addition to having a series of structural deficiencies, need to perform under liquefied and non-liquefied soil conditions. The structural deficiencies include the following. 1) The piles are not positively attached to the cap. Thus, there is limited or no up-lift capacity for the piles to resist overturning of the structure or moments in the footings. 2) There is no top or shear reinforcing in the pile caps. Again, this restricts the ability of the cap to resist moment. 3) The soils around the caps and supporting piles may liquefy. The foundations analyses are discussed in more detail in Appendix E.

This study looked at three foundation conditions. 1) The fixed base condition was analyzed to establish an upper limit of seismic force on the frame coupled with the minimum deflections. 2) A non-liquefied soils condition was analyzed. For ground motions with a 108-year return period, the structure will likely see maximum seismic shaking before the soil liquefies. The non-liquefied soil case reasonably reflects the foundation conditions the Viaduct frames are likely to experience during ground shaking to the level of the Expected Earthquake. 3) A liquefied soils condition was analyzed. Liquefied soils will establish a reasonable lower limit of seismic forces on the frame accompanied with the maximum deflections.

Although in seismic analysis it is often assumed that a fixed base gives the worst-case seismic demands on the frames, this practice does not address failure modes of the foundations themselves. A fixed base foundation analysis may mask the possible beneficial effects of softening of the structure that can increase the structural period and reduce the seismic force on the structure. Similarly, a fixed base analysis can ignore potential deficiencies identified in the foundations of the Viaduct, which can affect stability. Finally, a fixed base analysis can underestimate the structural deflections.

The following general analysis steps were used in this study. These procedures are described in more detail in the technical appendices to this report.

Structural analysis software Georgia Tech Structural Design Language (GT-STRUDL) (Version 29) was used to perform the pushover analysis. This latest version of GT-STRUDL provides an option to model flexure members with representative concrete and steel fibers with specific stress-strain relationships and limits during the formation of plastic hinges. These procedures track loads, load redistributions, component capacities, and curvature deformations of moment resisting joints during the lateral pushover analyses of the representative frames. Other brittle events such as column and/or joint shear failure are separately evaluated and accounted for in the bent's lateral load-deflection representation. The

GT-STRUDL analysis procedures do not track shear-related failures directly. See Appendix B for more detail.

For each cross section in the GT-STRUDL model, a custom section was developed for distributed plasticity analysis. The section was broken into a fine mesh for which concrete and steel fibers were defined. The concrete was treated as unconfined. Each steel fiber had a custom stress-strain curve to reflect the bars development length, buckling capacity, and splice capacities. By this method, the GT-STRUDL analysis procedure could account for the bars influence systematically. See Appendix C for more information regarding how the stress-strain curves were modified to account for bar development and splice issues. An independent check of GT-STRUDL plastic section analyses was performed using industry standard section analysis software Cross-Sectional Structural Analysis of Components (XTRACT). The distributed plastic procedures were also checked against the more classic concentrated plasticity pushover analysis, where the model is altered to substitute plastic hinges where distress occurs. Appendix G describes the model verification procedures in more depth. With the distributed plastic analysis, the flexure related structural deficiencies of the girders, columns, and pile caps were systematically developed for the pushover analyses of the selected bent.

The foundations at Bent 83 were modeled with non-linear lateral springs and compression-only vertical members to represent the piles with no tension connection to the pile cap. The software Deep Foundation System Analysis Program (DFSAP), developed for WSDOT, was used to determine the non-linear support capacities of the soil. DFSAP procedures account for the pile group effects as well as the effect of the pile cap as it is being pushed through the surrounding soil. Two sets of lateral springs were developed – one for non-liquefied soils and the other for liquefied soils. Compression-only elements were used to model the piles in the cap. Since the piles are not positively attached to the cap, it is conservative to assume the piles cannot take uplift. Compression-only elements allow the vertical loads and moment on the cap to be systematically transferred to those piles not in tension. This approach tracked the pile load and cap forces as the foundation rocked on the compression piles.

Shear capacities of the members and joints were handled by post-processing of the GT-STRUDL model's pushover analysis results. The GT-STRUDL software ran the pushover analysis until the structure either failed relative to moment capacity (sufficient hinge formations for collapse), crushing of the concrete, or sufficient lateral deflection (lateral foundation movements) to terminate the analysis. The resulting loads at each push increment were then used to evaluate the shear capacity and shear loads for the members and joints.

Member shear resistances are a function of load and ductility; the joint shear resistances are a function of the loads framing into the joint and the concrete capacity of the joint. Procedures from the Multidisciplinary Center for Earthquake

Engineering Research (MCEER) MCEER-06-SP10 (2006) and Priestley et al (1996), were used to evaluate member and joint shear capacities. The load levels where the column or joint shear capacities are reached were identified as collapse events, since the resulting brittle failure will undermine frame stability. Shear failure events therefore were used to truncate the structural capacity curves developed from the local pushover analyses.

The resulting load vs. deflection curves for the structures from the pushover analysis was converted to an equivalent SDOF system and evaluated by two different non-linear static procedures. One is the so-called N2 method based on the equal displacement rule presented by Fajfar, P. (2000). Eurocode 8, which specifically deals with seismic performance of bridges, uses an N2 method. The other is the Capacity Spectrum Method (CSM) presented in Federal Emergency Management Agency (FEMA) 440, (2005). MCEER-06-SP10 discusses CSM procedures relative to retrofits of bridges. Both these procedures adjust the elastic spectra to account for the ductility (N2) and damping (CSM) effects of the structure as it degrades relative to the damage characterized by the pushover analysis. Each procedure evaluated the structural performance relative to the Expected Earthquake response spectra. Depending on where the structural performance points plotted relative to the spectrum, it can be determined if a seismic ground motion level having a longer or shorter return period than the 108-year Expected Earthquake can result in frame instability with potential for collapse. See Appendix A for a more in-depth description of the non-linear static methodologies.

1.4 Findings

Three methods were pursued to estimate if a ground motion with a 108-year return period or less could initiate structural instability with potential for collapse of portions of the Alaskan Way Viaduct. First, the original SSRC data were plotted against the site specific Expected Earthquake response spectra developed in 2004 for the AWVSRP. Second, the results of the non-linear static analyses were evaluated relative to the structure's ability to survive the Expected Earthquake. Third, an estimate based on previous analyses was made regarding the extent of soil liquefaction relative to failure of the Alaskan Way Seawall, which could trigger a subsequent mass movement of soil that in turn could collapse the Viaduct. Each is briefly summarized below.

1.4.1 2001 SSRC Seismic Capacity versus 2004 Expected Earthquake Demands

The first method consists of plotting the original 2001 SSRC seismic capacity data for a selected section of the Viaduct against the 2004 site-specific Expected Earthquake response spectra. The SSRC evaluation in 2001 was based on regional seismicity from the United States Geological Survey (USGS) (Frankel et al. 1996). The regional USGS work used by the SSRC includes the effects of the Seattle Fault and other seismic sources as they were known in 1996. Based on the 1996 USGS data, SSRC estimated a 210-year return period for the seismic ground motion that could initiate collapse of the Viaduct. This is approximately a 1 in 20 chance of occurrence in the next 10 years.

As part of the AWVSRP work, Shannon & Wilson in the 2004 seismic ground motion study (Shannon & Wilson, Inc. 2004) included a local seismicity model that accounts for the Seattle Fault, the Tacoma Fault, and other seismic source zones. The Shannon & Wilson study, estimated seismic ground motions at the site using the knowledge of Puget Sound seismicity and tectonics as it was known in 2004. This study also developed the design spectra for the Expected Earthquake that is part of the AWVSRP design criteria. The Expected Earthquake is a ground motion that has a 50 percent probability of being exceeded in the 75-year design life of the Alaskan Way Viaduct replacement structures. This is a 108-year return period or approximately a 1 in 10 chance of occurrence in 10 years.

The SSRC analysis looked at the segment of the Viaduct for Bents 97 through 100 (Yesler Way to Washington St.). Their study indicated that the threshold seismic structural capacity for a typical frame along this portion of the Viaduct occurs at a spectral acceleration of 0.26g at a structural period of 1.5 seconds. The bents analyzed in the SSRC study straddle the site-specific ground motion Zones A and B defined in work by Shannon & Wilson, Inc. (2004). The SSRC threshold spectral acceleration is plotted on Figure 1.4 against the site-specific Expected Earthquake design spectra for Zones A and B (Shannon & Wilson, Inc. 2004).

As shown in Figure 1.4, the SSRC threshold falls slightly below the Zone B spectrum and above the Zone A spectrum. This indicates that according the SSRC threshold, ground motions in Zone B may be sufficient to trigger significant loss of component and frame capacity for bents in that zone, leading to frame instability with a potential for collapse of the structure. The bents in Zone A would have ground motions slightly below that to initiate frame instabilities and potential collapse. This comparison of threshold capacity and demand for Expected Earthquake ground motion levels suggest that a ground motion close to the Expected Earthquake 108-year return period can initiate structural instability.

1.4.2 Seismic Capacity versus Demand Analysis

The second method to estimate the seismic ground motion return period that could lead to structure instability and potential of collapse of the Alaskan Way Viaduct comes from analyses of a representative structural bent that supports the Viaduct.

Bent 83 was analyzed by non-linear static procedures to assess its seismic performance capability. The existing Viaduct bents are known to have deficiencies, stemming from the circa-1950 detailing practices that did not account for modern understanding of the way structures fail under earthquake loads and deflections.

Frame (above ground) deficiencies include the lack of confinement for the concrete; limited shear steel in the members and joints; low embedment length for the bars; and bar splices that may not develop full capacities. Foundation (below ground) deficiencies include the piles that are not positively attached to the cap, absence of the top reinforcement in the cap, and absence of the shear reinforcing in the cap. Thus, the pile caps have limited ability to resist uplift, moment, and shear for the base of the columns. The soil that surrounds the cap and provides lateral support can liquefy during or immediately after a seismic event. Soil liquefaction can lead to a drastic decrease in the lateral and vertical load-carrying abilities of the foundations.

Each deficiency was addressed in the analysis regarding its effects on the ability of the structure to withstand seismic loads and displacements. A pushover analysis was performed to develop a structural load versus capacity curve that reflects the way the capacity of the bent degrades for various levels of lateral force and deflection. The resulting capacity curves were converted to an equivalent SDOF system. Two non-linear static procedures using the SDOF system were then applied to compare the structural capacity against the seismic demand of the Expected Earthquake. These analyses were performed for several foundation conditions, including a fixed base, non-liquefied soil, and liquefied soil.

Pushover analyses show that the capacities are controlled by shear failures. For the fixed foundation, a joint shear failure occurred inside the column/footing joint. For the non-liquefied soil case, a member shear failure occurred in the pile cap, preventing the compression piles from carrying the column loads. In the liquefied soil case, the soil fails and large lateral movement of the structures are expected. The pushover curves show that the structure behaves fairly well regarding moment capacities. The moment capacity is terminated by crushing of the concrete, unless a soil failure occurs first. However, shear failure of the footing truncated the capacity curves and made the overall structural performance brittle. See Figure 1.5, 1.6, and 1.7 for the pushover capacity curves for fixed, non-liquefied, and liquefied foundation conditions.

As an earthquake damages the structure, the structure modifies and reduces the effects of the earthquake on the structure. The damaged structure becomes more limber and draws less seismic load. The act of damaging the structure dissipates energy. As the structure absorbs energy through damage, it dampens the earthquake-induced force demands. A reduction in stiffness and an increase in damping are beneficial to the survival of the structure; provided the components have ductile design detailing (absent from Alaskan Way Viaduct) to withstand deformations/damages beyond elastic limits.

The two non-linear static procedures used in this analysis tracked these beneficial effects by different methods. One method is an N2 technique, which used an equal displacement approach that tracks the load versus capacity of the structure through