POST-EARTHQUAKE EVALUATION OF THE ALASKAN WAY VIADUCT

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Figure 1, The Alaskan Way Viaduct

Abstract

The Alaskan Way Viaduct is a 2.2 mile long double-decked, reinforced concrete viaduct in Seattle, Washington. The viaduct was damaged in the 2001 Nisqually earthquake. The damage to the structure generally consisted of cracking of the transverse floorbeams and joints within each bent, and of the longitudinal edge girders. The cracking of the knee and crossbeam joints at Frame 100 was the most severe damage. This paper describes the post-earthquake evaluation of the structure, which included analyses to simulate the damage observed after the earthquake and rating analyses to determine the residual capacity of the structure. Retrofit and replacement options are described.

Introduction

The Alaskan Way Viaduct (Figure 1) is a 2.2 mile long double-decked, reinforced concrete viaduct carrying State Route 99 along the shoreline of Elliot Bay and past downtown Seattle. It is an important part of Seattle's road system, carrying approximately 110,000 vehicles per day. The 1950's vintage structure does not meet modern standards for earthquake resistant design. The viaduct is underlain by soft soils that are likely to liquefy during a major earthquake, and otherwise excite the structure near its fundamental period.

The viaduct contains several critical structural deficiencies [4]:

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- Both the upper and lower-story columns have inadequate confining reinforcement.
- The lower-story columns have inadequate transverse reinforcement and may fail in shear before reaching their flexural capacity.
- The joints are inadequately reinforced and may fail in shear before the adjacent members hinge.
- The footings could fail in shear if the lower-story columns do not.

On February 28, 2001 the Nisqually earthquake struck the Puget Sound region of Washington State. The epicenter of this magnitude M_w=6.8 earthquake was located in the Nisqually Valley about 12 miles northeast of Olympia, WA and about 35 miles southwest of Seattle, WA [3]. The Alaskan Way Viaduct was significantly damaged in this event. This paper describes the damage to the viaduct, the post-earthquake evaluation of the structure by the Washington State Department of Transportation (WSDOT) [8, 9], the immediate repair of the structure by the WSDOT, and the basis of the State's recommendation to replace the structure [6].

Damage from the Nisqually Earthquake

The most severe damage to the viaduct during the Nisqually earthquake was to the structural unit comprising Bents 97-100, near S. Washington Street. The unit is in the part of the viaduct designed by the City of Seattle Engineering Department (SED); other parts of the viaduct were designed by the Washington State Department of Transportation (WSDOT). The damaged unit is approximately 222 feet long. It consists of four transverse frames linked together by edge girders spanning longitudinal between the columns. Both the upper and the lower deck slabs are supported on a grid of stringers and intermediate floorbeams that transfer load to the frames and the edge girders. The portion of the viaduct designed by the SED is characterized by relatively heavy stringers that transfer load directly to the frames, whereas the portion of the viaduct designed by the WSDOT has relatively heavy intermediate floorbeams that transfer load to the longitudinal edge girders.

Figure 2, Damage to Bent 100, East Column, Upper Deck Joint

Figure 3, Damage to Bent 97, East Column, Upper Deck Joint

Bents 97-100 are largely independent of adjacent structural units. The end bents share a common footing with the end bents of the adjacent units—i.e., the end bents are “split bents.” Except for these footings the bents are transversely independent. Longitudinally, the units can pound into each other during an earthquake. Bents 97-100 are on an 800’ radius, whereas the bulk of the Alaskan Way Viaduct is straight. The curvature of this unit may have contributed to the greater damage suffered at this location.

The damage to the structure consists of cracking of the transverse floorbeams and joints within each bent, and of the longitudinal edge girders [7]. The cracking of the joints was the most severe damage. The upper, east knee joint of Bent 100 was badly cracked and spalled and the reinforcement within this joint was exposed. Some of this reinforcement was fractured. The damage to the joint is shown in Figure 2. The upper, east knee joint of Bent 97 was also cracked. The damage to this joint is shown in Figure 3.

The damage to the structure was assessed from photographs made available by the WSDOT[4]. From this assessment, the damage to the joints of the structure is summarized in Table 1.

Table 1, Damage to Joints

Bent	**Joint**	**Joint**	
		West	**East**
97	Upper	No cracking	Moderate cracking, see Figure 3
	Lower	No cracking	Minor cracking
98	Upper	No cracking	Moderate cracking
	Lower	Minor cracking	No cracking
99	Upper	Minor cracking	No cracking
	Lower	No cracking	No cracking

[4] There was no comprehensive survey report of the damage to the structure.

Table 1, Damage to Joints			
Bent	**Joint**	**Joint**	
		West	**East**
100	Upper	No cracking	Severe cracking and spalling of concrete; some fracture of reinforcement; see Figure 2
	Lower	No cracking	Minor cracking

The unit is also displaced to the east. The measured drift of the structure was approximately 3 inches at the top of Bent 100 and 1½ inches at Bent 97 [7]. Reportedly, approximately one-half of this displacement predates the Nisqually earthquake. The other half was presumably earthquake damage.

The reduced strength and stiffness of the joints may alter or limit the path for vertical loads acting on the structure. Also, the lateral drift of the structure implies that there are residual internal forces in the bents, and these may reduce the capability of the structure to carry vertical loads. Because of these concerns, the structure was closed to truck traffic.

WSDOT requested that TYLI model the damage to the structure and rate the structure for dead and live loads in light of the damage. The starting point for the rating was the damage observed in the field. Our work did not include dynamic analyses to investigate the response of the damaged structure to future earthquakes.

Approach

The general approach to the problem was to include the damage to the structure in an analytical model, and then apply dead and live loads to the damaged model in order to find the demands on the structure. The strategy to accomplish the analysis was as follows:

1. A three-dimensional beam and shell element model of the unit comprising Bents 97-100 was constructed. The bents, including the crossbeams and the columns, were modeled with nonlinear moment-curvature beam elements. The joints were also modeled with moment-curvature elements to simulate the joint damage and stiffness.

2. Dead load was applied to the model.

3. A mass-proportional lateral load was applied to the model and then removed (simulating the earthquake). This was done to reproduce the permanent offset observed in the field.

4. Factored dead load was applied to the model. A load factor of 1.2 was used in accordance with the AASHTO “Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges” [2].

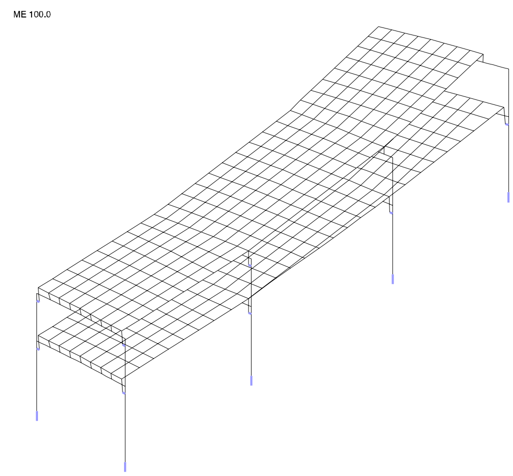

Figure 4, ADINA Model of the Structure

5. Factored truck loads were applied to the model, together with the corresponding centrifugal (lateral) loads.

   5.1. HS20 truck loads were used with a 1.3 design-level impact factor.

   5.2. The load factor was taken from the "Guide Specifications for Strength Evaluation…" [2]. For heavy volume roadways with "significant sources of overloads without effective enforcement," the Guide Specifications specify a load factor of 1.8. This represents the upper limit of load factor, which in some measure accommodates latent damage and deterioration due to seismic loads that may not be included in the model.

   5.3. Four load patterns were analyzed on each of Bents 97 & 100.

6. The capacity of the structure was computed in accordance with the 16^{th} edition of the AASHTO Standard Specifications.

<u>Modeling</u>

A three-dimensional model of the structure was made using the ADINA general purpose finite element program [1]. This program performs geometrically nonlinear analysis and it has nonlinear elements able to model the flexural hinging of beams and other elements. The ADINA model of the structure is shown in Figure 4. The curvature of the structure is faithfully reproduced in the model. The model includes the widening of the lower deck at Bent 97, and the outrigger bent from the upper deck.

The crossbeams and columns were modeled with ADINA moment-curvature beam elements to simulate the flexural hinging of those elements. The reinforcement of the crossbeams and columns is highly variable along their length and height, with frequent bar cutoffs. The consequent variation of strength and stiffness is reproduced using the moment-curvature elements.

The upper and lower decks were both modeled with continuous grids of beam and shell elements. The flexural properties of each deck were lumped into the beam elements and the membrane stiffness of each slab was lumped into the shell elements. The footings were modeled with rotational springs acting about the longitudinal and transverse axes. These were computed from the axial stiffness of the H-piles supporting the footings, assuming that the piles are tip-supported.

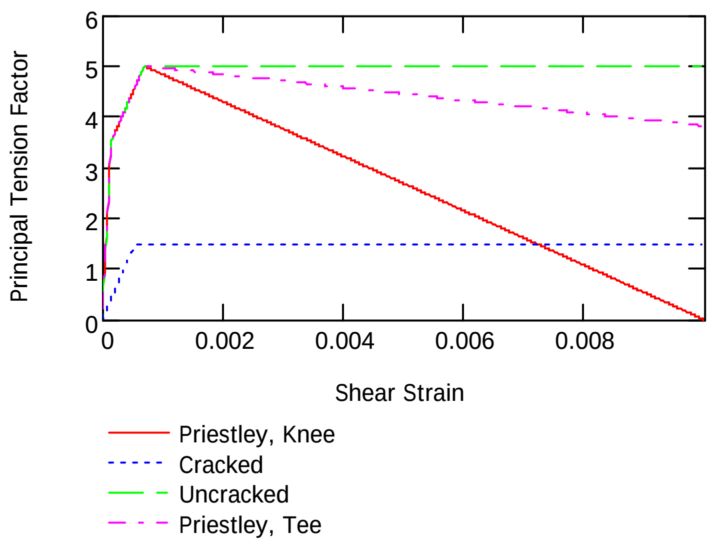

Figure 5, Degradation of Joint Principal Tensile Strength vs. Shear Strain, Priestley, et al. [5]

The modeling of the joints was motivated by a relationship between joint principal tensile stress and shear strain developed by Priestley, et al. [5] for the seismic assessment of bridges. This relationship is shown in Figure 5. The relationship models the initial stiffness of a joint, the cracking of the joint at a shear strain of 0.00015, the consequent reduction of stiffness, and the deterioration of the strength of the joint beyond a strain of 0.0007.

Joints which are uncracked were modeled with this relationship, except that because ADINA cannot model degradation of stiffness, the modeling assumed the strength to be constant beyond a shear strain of 0.0007. This is the model labeled “uncracked” in Figure 5. Of course, the model is only realistic if the strain is not much larger than 0.0007. The “uncracked” model was only used for joints that were observed to be uncracked, and the analysis was monitored to verify that the joint shear strains were indeed small.

Joints which were observed to be cracked were modeled with the relationship labeled “cracked” in Figure 5. The initial part of this relationship reproduces the stiffness of a cracked joint from the Priestley model. Because ADINA cannot model degradation of stiffness, the model assumes that the joint shear strength is constant beyond a certain strain. The plateau strength must be assigned based on the peak shear strain occurring in a joint, e.g., to intersect the Priestley model at the peak shear strain. An iterative analysis was required to obtain meaningful results. The plateau strength is the principal tensile strength of the joint and is characterized by a factor n:

$$p_t = n \cdot \sqrt{f_c^{'}}$$

The plateau strength of each joint was initially assigned based on judgment, from the observed cracking of the joint—see Table 1. Upper, knee joints were assigned a plateau strength corresponding to $n = 3.5$ or less, since this type of joint degrades relatively quickly beyond a shear strain of 0.0007. Lower, tee joints were assumed to be capable of developing a principal tensile stress corresponding to $n = 5.0$ since this type of joint degrades less quickly. The peak shear strains occurring in the analysis were then checked to determine the strength according to the Priestley model. For all of the joints, the assigned plateau strength was found to be compatible with the shear strains observed in the analysis.

The modeling of each joint is summarized in Table 2. The modeling closely parallels the observed damage summarized in Table 1. The factors n are related to the severity of cracking and are also consistent with the peak shear strains observed in the analysis.

Table 2, Joint Modeling

Bent	Joint	Joint	
		West	**East**
97	Upper	Uncracked	Cracked; n=1.5
	Lower	Uncracked	Cracked; n=5.0
98	Upper	Uncracked	Cracked; n=1.5
	Lower	Cracked; n=5.0	Uncracked
99	Upper	Cracked; n=3.5	Uncracked
	Lower	Uncracked	Uncracked
100	Upper	Uncracked	Hinge; i.e., n=0.0
	Lower	Uncracked	Cracked; n=5.0

In ADINA, the relationship between principal tensile stress and shear strain was modeled with moment-curvature beam elements by first relating the principal tensile stress to the shear stress (under an assumed vertical stress due to dead load) using Mohr's circle, and by then relating the shear stress in a joint to the moment acting on it using an equilibrium relationship. Shear strain was considered to be equivalent to a joint rotation and this was related to curvature according to the dimensions of the joint. At each joint of the ADINA model the moment-curvature element extended from the column centerline to its face.

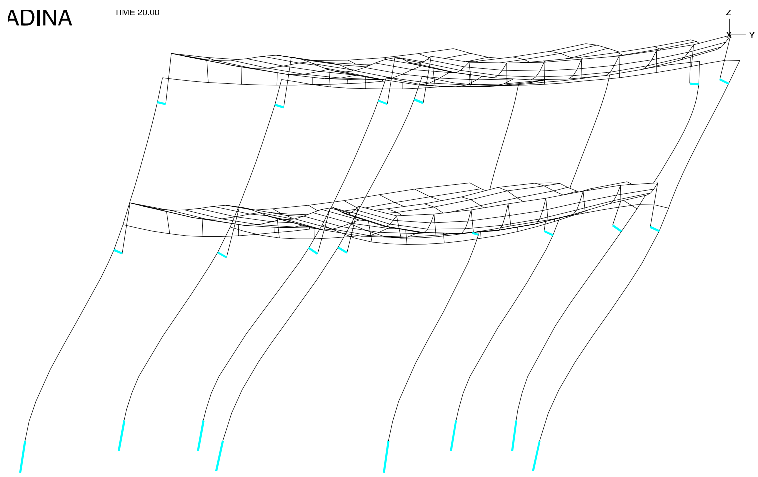

Figure 6, Deflected Shape under Mass-Proportional Lateral Load (100 times scale)

<u>Analysis</u>

The first step in the analysis was to apply the dead load and solve for the corresponding deformed position. Then a mass-proportional lateral load was applied to the model in order to reproduce the damage observed in the field, specifically the eastward lean of the structure. Figure 6 shows the deflected shape of the model under a mass-proportional lateral load corresponding to an acceleration of 100 inch/sec/sec = 0.26g. This was the maximum lateral load that the model could support without collapse. When the lateral load was removed the structure sprang back to the residual deflections shown in Table 3:

Table 3, Residual Deflections

Bent	Deflection, inch	
	Lower	Upper
97	0.35	0.63
98	0.49	0.92
99	0.69	1.24
100	0.80	1.49

These deflections are consistent with the observed deflections of the structure, which are approximately 3 inches at the top of Bent 100 and 1½ inches at Bent 97 [7], if one takes at face value the reports that approximately one-half of the measured displacement predates the Nisqually earthquake, and that the other half is earthquake damage. Unfortunately, the documentation of the observed displacements is incomplete. Also, no account has been made of permanent rotations of the footings, either in the field observations, or in the analysis. Even small rotations of the footings would produce measurable displacements.

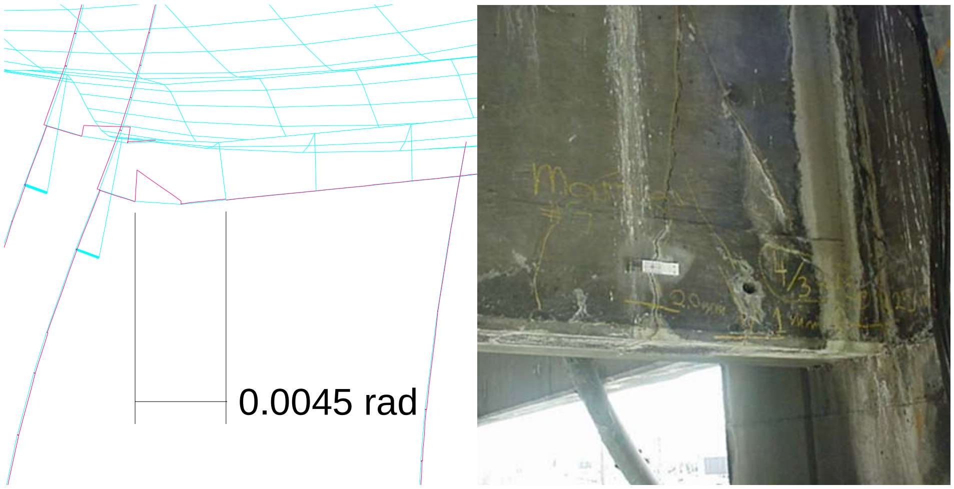

Figure 7, Curvature of Bent 100, Lower Crossbeam, in Damaged Condition

Figure 7 shows the curvature of the lower crossbeam in the damaged condition (after removing the lateral load), plotted on the deflected shape of the structure. The curvature is greatest in the element nearest the column since this is the weakest section (in positive bending). The curvature particularly tends to concentrate at the face of the column, where the moment is largest. A more reliable measure of the damage is the total rotation of the plastic hinge, which is 0.0045 radians. The average curvature of the crossbeam is then about 0.000075 /inch, which is consistent with significant cracking of the crossbeam. The observed cracking of the crossbeam is shown in Figure 7 also.

Subsequent to the application and release of the lateral load, an increment of dead load equal to 0.2 times the dead load was applied to the model. The total dead load acting on the model was then the factored dead load, equal to 1.2 times the dead load.

<u>Rating</u>

On top of the factored dead load, factored HS20 truck loads were applied to the model in accordance with the approach outlined above.

The flexural and shear capacity were computed in accordance with the 16th edition of the AASHTO Standard Specifications. The moment capacity was taken to be the nominal capacity times a resistance factor of 0.90. The shear capacity included a resistance factor of 0.85. The shear strength of the concrete was taken to be simply $v_c = 2\sqrt{f_c'}$. The material properties used were the nominal specified properties, i.e., $f_y = 36\ \text{ksi}$ and $f_c' = 4200\ \text{psi}$. No account was made of the possible increase in strength of the concrete with age above the specified value.

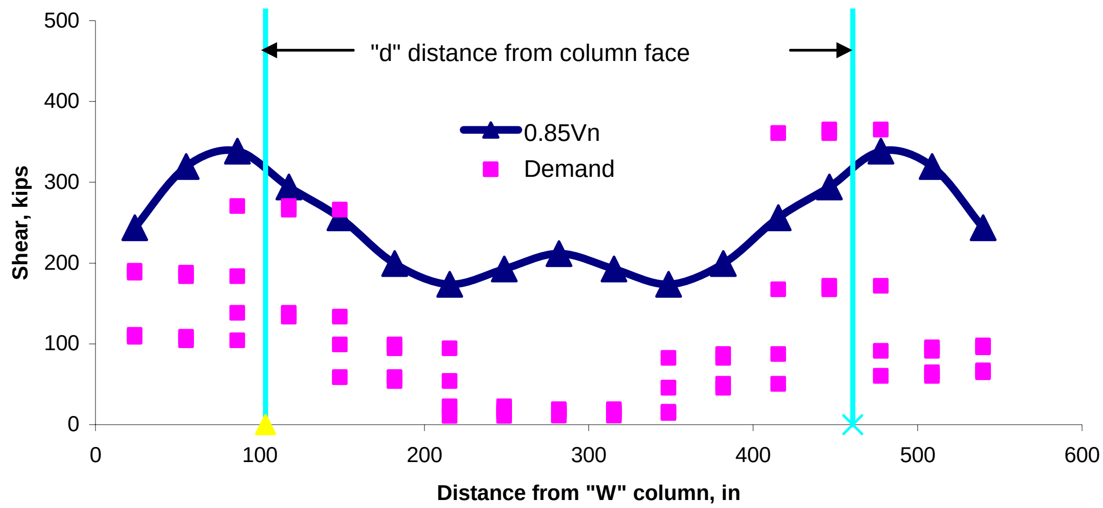

Figure 8, Bent 100, Upper Crossbeam Shear Diagram

In all cases the flexural demands were less than the calculated capacity. This was not the case for shear, however. Figure 8 compares the shear demands on the upper crossbeam of Bent 100 with the capacity of that element. The cross-beam is overloaded in the vicinity of the failed hinge—on the east side of the structure.

Since the structure cannot carry the full factored live load, it is useful to compute a rating factor:

$$RF = \frac{\phi R_n - \gamma_D D - \Delta}{\gamma_L L(1+I)} \text{ where}$$

φ = a resistance factor,

R_n = the nominal resistance (strength),

γ_D = the dead load factor (1.2),

D = the dead load effect,

Δ = the effect of damage to the structure, i.e., a load due to redistribution of forces,

γ_L = the live load factor (1.8),

L = the live load effect, and

I = the impact factor (1.3).

For the critical upper crossbeam of Bent 100, the rating factor for HS20 loads, for shear, is:

$$RF = \frac{(0.85)(301) - (1.2)(91) - 49}{(1.8)(95)(1.3)} = 0.44\,.$$

This result strongly suggests that trucks should be prevented from using the structure until the earthquake damage is repaired and other general rehabilitation measures are completed.

Post-Earthquake Repairs

The immediate response of WSDOT after the earthquake was to shore up the transverse floorbeams and install a horizontal steel tie-rod to couple the damaged unit to the adjacent unit across the joint at bent 100. This action reduced the lateral deflection under live load at pier 100, but increased that deflection at pier 97. Therefore, the same type of transverse rod was installed at bent 97. Owing to a concern about lateral strength of this unit, and the lateral live load on the 800 ft curve, WSDOT proceeded to install diagonal bracing at bents 98 and 99, designed to resist all lateral loads in the unit for up to 10% of dead load. This repair was termed “Phase 1.” WSDOT designed a “Phase 2” repair to include beam reinforcement by fiber strengthening for flexure and shear in the transverse frame floorbeams.

The analytical evaluation showed that while the diagonal bracing installed under the immediate “Phase 1” repair was sized for the equilibrium conditions of lateral load, these braces were too flexible to pick up load from the stiffer concrete frame at a rate that would preclude further damage to the concrete. Therefore, the braces were deemed unacceptable as a repair solution, and a “Phase 3” repair was planned, which included removing the diagonal braces and replacing them with an alternative strengthening method. While this work was underway, it appeared that a repair of the upper knee joints at bents 97 and 100 through the use of drilled in reinforcing bars could restore adequate lateral resistance to the unit.

Retrofit and Replacement Options

The viaduct faces a variety of threats, and the risk of the occurrence of each increases over time. The primary structural threat is that the ground shaking from a seismic event may cause a variety of structural elements to fail. In addition, the soil under the viaduct is very poor and could liquefy, adding the threat of sufficient movement at the foundations to induce collapse of local viaduct frames.

In order to evaluate the options for addressing these threats, some standard level of acceptable risk must be established. First it should be recognized that reducing to zero the risk of collapse of any structure is impossible, even if the structure is new and is designed and erected to modern standards. Uncertainties over the size of the earthquake loading and the strength of both the foundations and the structure mean that some risk, however small, will always be present.

The 500 year return level of earthquake has been applied by the engineering profession as a minimum basis for design, and is written into past bridge specifications. Therefore, this event level was adopted by the WSDOT as the minimum acceptable risk for the various options for dealing with the old viaduct. After evaluation of risks and site conditions, the WSDOT considered three engineering options:

1. Option 1 was described as “Repair Only – Return to pre-Nisqually Earthquake Condition”. It consists of performing the repairs planned under the WSDOT’s Phase 1, 2 and 3 contracts, which address exclusively bents 97-100. These were the parts of the structure that were most heavily damaged during the Nisqually Earthquake of 28 Feb 2001. Under this option, no other action is taken apart from routine maintenance. This is clearly not a sound long-term solution because, under it, the viaduct continues to deteriorate and may collapse under a moderate earthquake only slightly larger than the Nisqually event. The 10% risk of exceedance is surpassed by 2021, and even earlier with general deterioration of the structure due to age and traffic demands.

2. Option 2 was described as “Immediate Replacement”. In this option, planning for replacement should start immediately, with the intent of having the new structure open to traffic in approximately 10 years. This duration of planning and construction results in a 5% chance of exceeding the earthquake assumed to cause collapse of the viaduct within the time for construction (this being caused by the 210 year event). As this process is delayed, the risk of having a moderate earthquake that could collapse the structure increases.

3. Option 3 was to retrofit the viaduct to full current standards, with the intent of creating a 50-year design life. This retrofit should be complete within 10 years, subject to the same issues and limitations for time exposure as option 2. Based on experiences in California, this level of retrofit is expected to have costs on the order of a new structure. The deck system would still be aged, and maintenance costs for the deck would likely increase with time.

The option to retrofit only the foundations for a 20 year life span, with the intent of having a replacement structure open to traffic within 20 years, was rejected as an alternative. This approach addressed only one of the risk factors with the current design, that being the risk of soil liquefaction that could fail the foundations. This option, like option 1, does not address the full spectrum of seismic risks of structural failure. Therefore, while it does deal with a major site risk, it does not alter the risk profile for the viaduct as a whole. Since this option did not address both the structural and the foundation risk, it was not included in the summary table of options.

The Alaskan Way Viaduct is approximately 50 years old and must be regarded as nearing the end of its useful life. Signs of age are manifest in functional obsolescence (e.g. narrow traffic lanes), in corrosion and fracture of reinforcing steel, and in cracking of the concrete. These characteristics detract from the structural integrity of the viaduct, and therefore increase the risks to the traveling public. Quite apart from the deterioration, the viaduct suffers from a large number of deficiencies in the structure and the foundation systems that leave the entire structure vulnerable to collapse in an earthquake event substantially less severe than the event currently considered as a minimum design standard for new bridges.

The major sources of uncertainty with seismic performance of the current viaduct include the size and timing of future earthquakes, the soil characteristics, the expected performance of the many inadequate structural details, and the extent of the deterioration caused by aging. All of these factors were considered by a Structural Sufficiency Review Committee appointed by the WSDOT to review the work presented in this paper and other work, and to recommend future action relative to the viaduct. This committee considered a range of options to address the seismic safety of the Alaskan Way Viaduct. The options ranged from a traditional “no-build” to full replacement of the viaduct. The recommendations of the committee were [6]:

1. The Alaskan Way Viaduct should be replaced with a new structure as soon as possible. This recommendation arises from evaluation of the risks and costs of each of the options considered.

2. The Committee did not recommend retrofitting the viaduct as a long-term solution. The estimated costs of retrofit would be similar to the estimated costs for replacement, and the Committee believed that replacement offered far greater value and reliability that comes with a completely new structure.

3. After repairs for deterioration and all damages incurred in the Nisqually earthquake, the Alaskan Way Viaduct may be operated at an elevated level of risk until replacement of the structure is completed within the recommended time frame. Control of operations must still be subject to the normal limitations for inspecting, maintaining and load rating of similar structures.

Summary

The Alaskan Way Viaduct was damaged in the 2001 Nisqually earthquake. The damage to the structure consisted of cracking of the transverse floorbeams and joints within each bent, and of the longitudinal edge girders, and perhaps further damage to foundations. The cracking of the joints was the most severe damage. Inelastic analysis of the most heavily damaged unit of the viaduct—comprising Bents 97-100—was performed to simulate the damage. The damaged structure was then rated to determine is remaining load-carrying capacity. The rating of the cross-beams for shear was found to be less than one for HS20 loads. Consequently, trucks were prevented from using the viaduct.

The viaduct is near the end of its useful life, and has many structural and functional problems that make retrofit of the viaduct a questionable investment. A Structural Sufficiency Review Committee, appointed by the WSDOT to study retrofit and replacement options, recommended that the viaduct be replaced. Replacement bridge and tunnel options are now being considered by WSDOT.

Acknowledgements

This paper depends heavily on the work of the Structural Sufficiency Review Committee appointed by the WSDOT [6] to serve as a peer review panel for both the immediate repairs and long-term evaluation of the Viaduct. This committee consisted of John H. Clark, PhD, PE, SE; Ben C. Gerwick, PE; David Goodyear, PE, SE; Paul Grant, PE; Robert Mast, PE, SE; and John Stanton, PhD, PE. The work of the committee is hereby acknowledged. The material presented in this paper, and its conclusions, remain the responsibility of the authors, however.

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