T·Y·LIN INTERNATIONAL

Alaskan Way Viaduct
Report of the
Structural Sufficiency Review Committee

June 28, 2001

TABLE OF CONTENTS

1.0 Executive Summary

The February 28, 2001 Nisqually Earthquake was a wake-up call for engineers and planners throughout Puget Sound. The earthquake hypocenter was deep, and the duration relatively short, sparing the Region from more significant damage than that incurred. However, that damage that did occur now serves as a focal point for improving our transportation infrastructure, and points us toward targets of the greatest priority for seismic safety. One of those targets is the Alaskan Way Viaduct, which experienced moderate shaking, yet incurred significant damage to a structural unit at Washington Street.

After the Nisqually Earthquake, the Washington State Department of Transportation Bridge and Structures Office (WSDOT) formed a Committee of independent engineers to serve as a peer review panel for both the immediate repairs and long-term evaluation of the Viaduct. The Committee title stems from our objective – the Structural Sufficiency Review Committee. The objective was to review the existing data on the Viaduct, and offer our recommendation to WSDOT on the matters of immediate repair of the Viaduct and longer-term recommendations for seismic retrofit or replacement. The Committee's scope has been limited to seismic safety and structural design of the existing Viaduct. The Committee has not addressed general transportation planning or capacity issues related to the Viaduct, nor have we considered what structure or configuration might be most appropriate for a replacement or retrofitted Viaduct.

The Committee has reviewed existing data from prior studies of the Viaduct, recent inspection reports and service load ratings, geotechnical reports, and reconnaissance reports from the Nisqually earthquake. In addition, the Committee has evaluated the general design and condition of the Viaduct using available engineering plans and historical data, with reference to contemporary codes and standards for bridge design. A significant feature of current codes is a linkage between the probability of occurrence of earthquake level and structural design requirements. WSDOT's current standard for earthquake level is based on ground shaking with a 10% chance of exceedance in 50 years.

Our Committee reviewed prior recommendations and the priority listings for retrofit of the Viaduct. We concluded that a piecemeal, item by item approach, is unwise. The retrofit of individual members inevitably changes their stiffness and affects the behavior of the other members. We therefore believe that any structural retrofit solution should be a complete retrofit of the structure (described in item 3 of 6.2.4), which would then permit a valid analysis and evaluation of the retrofitted structure as a whole.

The Committee focused on a comprehensive engineering review within the reach of our scope, which began with setting minimum acceptance standards for structural performance, and included evaluation of relative cost efficiency in achieving these minimum engineering standards. Using the basic seismic design standard of 10% chance of exceedance in 50 years, the three options were defined in both the short and long term context, with the short term being a transition period necessary to reach each objective, and the long term being characterized by the residual earthquake risk of the respective alternative.

This information has served as a basis for our opinion on the following options for the Viaduct:

TABLE 1

Options and Risks

Item	Decision/Option	Near Term Risk	Long Term Risk
1	Repair 97/100 only (return to pre-Nisqually EQ condition)	Same as current	Risk of exceedance = 210 yr event (1 in 5 chance of exceedance in 50 years, even without deterioration)
2	Replace as soon as financing, planning and engineering can be completed	10 years to complete (risk ~ 1 in 20 chance of exceedance in 10 years for 210 year event)	Replaced (standard risk 1 in 10 chance in 50 years)
3	Full retrofit to current standards	10 years to complete. (Risk same as item 2)	Replace within 50 years.

As a result of our review, the Committee recommends that WSDOT proceed to replace the Viaduct, which is option 2 in Table 1. This recommendation is based on our conclusion that even though a comprehensive seismic retrofit might achieve a level of safety comparable to a new structure, the eventual deterioration of the current structure due to ageing would exact a greater sum of financial resources for maintenance and be less reliable than a new structure built to current seismic design standards. The least cost alternative – Option 1 – does not begin to satisfy the seismic risk criteria for our study, and will leave the risk of collapse due to earthquake at a level more than twice the risk for a new Viaduct.

Submitted by the Committee

John H. Clark, PhD, PE, SE

Ben C. Gerwick, PE

David Goodyear, PE, SE

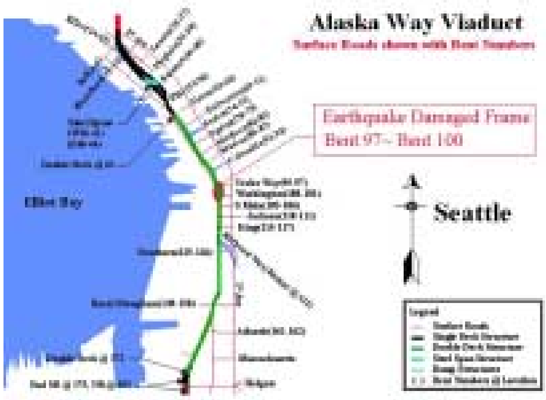
Paul Grant, PE

Robert Mast, PE, SE

John Stanton, PhD, PE

2.0 Introduction

The Alaskan Way Viaduct is a 2.1 mile long double-decked, reinforced concrete Viaduct carrying State Route 99 along the shoreline of Elliott Bay past downtown Seattle. It is a vital part of Seattle's highway system, carrying approximately 100,000 vehicles per day. The Viaduct was constructed in two major phases. The first was designed by the Seattle Engineering Department, and built in 1952. The second section, to the south, was designed by the Washington State Department of Transportation (WSDOT), and built in 1956. The design details for the two sections differ, however neither section meets modern standards for earthquake resistant design. In addition, limited segments along the Viaduct are exposed to risks related to the Seattle seawall structure, which in the event of major soil liquefaction in an earthquake, may allow lateral spreading of the soil that provides support to the pile foundations on the Viaduct. This seawall structure is an additional vulnerability that must be addressed when assessing the Viaduct.



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 Delta about 12 miles northeast of Olympia, WA and about 35 miles south of Seattle, WA.

After the February 28 earthquake, the WSDOT Bridge office conducted a drive-through inspection of the structure to assess major structural condition. The most notable damage was to a curved bridge unit adjacent to Washington Street, comprised of bents 97 – 100.

Damage at Bent 100

Immediately after the earthquake, the WSDOT Bridge office addressed the structural safety issues of the Viaduct, first dealing with the immediate issue of structural damage, and then addressing a longer-term solution to the seismic risk issues surrounding Viaduct operations. As part of the latter effort, the Washington State Department of Transportation Bridge and Structures Office formed an ad hoc Committee to review the structural sufficiency and seismic safety of the Viaduct, and to present recommendations regarding the following issues:

1. Short-term operation of the Viaduct
2. Vulnerability of the Viaduct in future earthquakes
3. Viability of overall seismic retrofit as a strategy for the Viaduct vs. Viaduct replacement

The Alaskan Way Viaduct Structural Sufficiency Review Committee (SSRC) first convened on May 10, 2001. The Committee's review of seismic safety is based on the standard of life safety for those using the mainline portion of the Viaduct. The Committee did not explore these issues for the ancillary structures or connecting ramps. This life safety approach is in contrast to one that deals with repairable damage to facilitate a rapid return to operation after an earthquake. The latter is a more demanding standard for structural design, and one that would more reasonably apply to the design of a new structure rather than to the retrofit of an old facility like the Viaduct. In order to maintain a direct comparison between options, all reference within this report is to the life safety standard, rather than to a repairable damage/operations standard.

In what follows, the 3 issues noted above are reduced down to 2. This reduction is a recognition that the Viaduct will need to be replaced in due course, if only for age and general deterioration. The operative questions then become when the Viaduct should be replaced, and whether or not an interim seismic retrofit is warranted before the time for replacement in order to address earthquake risks.

2.1 Standard for Evaluation of Risk

The Committee set the measure of seismic risk and reliability for the Viaduct according to generally accepted levels of risk for similar structures. The current standard in Washington for new bridge design is to prevent collapse of a structure for a seismic event having 10% chance of being exceeded in 50 years. This event is also defined by a Return Period or Mean Recurrence Interval (MRI), which is the reciprocal of the annual probability of the event. The return period for the current design event is nominally 500 years (by calculation, 475 years). Over a 75-year lifetime, this event relates to a 15% chance of having a seismic event that would exceed the design requirements for the structure. The Committee used the 10% exceedance in 50 years as a standard of acceptable risk. Since the cumulative probability increases with time, the reasonable time to accomplish either retrofit or replacement strategies was determined using this risk criterion.

The Committee sought to characterize both the existing structure and the seismic hazards in order to develop recommendations for this report. Given the limited time for this report, the Committee did not undertake the degree of engineering studies needed to produce independent data for these characterizations, opting instead for the use of existing data and outside sources for input to our reviews. In order to evaluate the vulnerability of the Viaduct and address the issues posed to the Committee, we developed the following engineering baseline from current data and our analysis of seismic hazard at the site:

1. The recent Nisqually earthquake event represents an earthquake with return period of 150 years for the Alaskan Way site.
2. The evaluations of the existing Viaduct, coupled with assessments of damage after the Nisqually event, place the threshold earthquake event for collapse of major portions of the Viaduct at a return period of 210 years. This return period was derived from a non-linear ultimate displacement (pushover) analysis performed on Unit 97-

100 that indicated a threshold spectral acceleration of .26g at the computed period of 1.5 seconds would reach the structural limit for the typical Viaduct frame.

3. The evaluations of the existing soil conditions and performance of the adjacent Seattle sea-wall structure in the Nisqually event suggest that widespread liquefaction sufficient to endanger the stability of the seawall and Viaduct would have a return period of about 350 years.

2.2 Other Studies

The Viaduct has been the subject of numerous studies. Immediately after the 1989 Loma Prieta earthquake in California caused the collapse of the Cypress Viaduct structure, WSDOT initiated studies of the Alaskan Way Viaduct out of a concern that a similar fate may be in store should a major earthquake affect Seattle. WSDOT began structural reviews in 1990, and carried out additional studies through 1996, including a comprehensive seismic risk assessment conducted for WSDOT in 1995/6 by the University of Washington (Refs 1, 2, and 3), and a later study by WSDOT on the Seattle seawall (Ref 4). The Committee also met with the authors of these studies in order to obtain their opinions on how their studies might relate to the Committee's work.

There are currently several parallel studies being conducted on the Alaskan Way Viaduct. The Washington State Office of Urban Mobility (OUM) implemented a Commission directive to study the Viaduct prior to the February earthquake. That study was intended to review global transportation and planning issues pertaining to the Viaduct operation, retrofit, replacement or removal. OUM has recently expanded their program to a full NEPA Environmental Impact Study (EIS) in order to more fully assess the alternatives related to the Viaduct, and to be prepared to act on any preferred alternative that emerges from the EIS.

3.0 Project Description

The Alaskan Way Viaduct runs from Holgate Street on the south to Mercer Street on the north, a length of approximately 2.1 miles. The majority of the Viaduct is a two level structure; with the upper northbound level serving up to 4 lanes of traffic, and the lower southbound level serving 3 lanes of traffic. This report addresses only that two level portion of the viaduct. Numerous ramp structures intersect the Viaduct, but they also are not within the scope of this report.

The northern portion of the Viaduct was designed by the Seattle Engineering Department, and completed in 1952. The southern portion of the Viaduct was designed by WSDOT, and completed in 1956. While the internal details of the two sections differ, they are both comprised mainly of 3 span monolithic reinforced concrete units. Split piers in between units accommodate thermal expansion. The typical details for the mainline units are shown in Appendix C.

Design and construction practices of the day (c. 1950) did not include special reinforcing details for earthquake resistance. The provision for earthquake loads was similar to the static lateral analysis then provided for wind loads. The level of lateral load used for earthquake design of the WSDOT section is reported to have been 10% of the gravity dead loads. While less than required today, this level of lateral load was nevertheless larger than the corresponding load used on other structures of the time, including the Cypress Viaduct in California (see below). Perhaps the greatest practical difference between contemporary design practice and that of the 1950's is in the reinforcing details for development of reinforcing bars and the lateral confinement steel installed to prevent the brittle concrete matrix from bursting under the extreme strains of earthquake loading. Since reinforced concrete is a composite of brittle concrete and ductile steel reinforcing, survival of the concrete sections depends upon the ability of the concrete to “hold” the reinforcing within the matrix. Thus, once lateral confinement of the concrete is lost, reinforcing bars pull out of the matrix, and strength is lost.

3.1 Comparison of Alaskan Way Viaduct with California’s Cypress Viaduct

The 1995/96 University of Washington studies were conducted to investigate the seismic vulnerability of the Alaskan Way mainline structures. This effort was prompted by the catastrophic collapse of the Cypress Viaduct in Oakland, California during the 1989 Loma Prieta Earthquake, a magnitude 7.1 event in the San Francisco Bay area. The Cypress Viaduct was a double-deck structure that was constructed between 1953 and 1957, the same general time frame as Alaskan Way. The portion that collapsed in the earthquake was supported on piles in an area of coastal fill, placed over natural bay mud. However, this is where the similarity between Alaskan Way and Cypress Viaducts ends.

The Alaskan Way Viaduct has two basic reinforced concrete framing systems. In the northern mainline, the three span units consist of monolithic transverse frames, and haunched interior beams that carry the roadways to the frames. The outside fascia girders are lightly reinforced, and do not have major duty in the gravity system. The southern mainline also has three span units, but the roadway loads are carried transversely to the outside fascia girders, and from there back to the monolithic transverse frames. Therefore, the outside fascia girders are the main longitudinal element for the southern section. Reinforcing details differ between the two sections, as do foundations. However the common element of the two basic designs is that there is monolithic reinforced concrete construction, with thermal expansion and contraction accommodated through split piers at the end of each 3 span unit. As a result, there are no sliding joints or hinges within the typical mainline structure.

In contrast, the Cypress Viaduct combined both reinforced concrete and prestressed concrete construction. While the transverse frames were generally monolithic in appearance, they included concrete hinges and shear keys to allow for shortening of the prestressed sections, and in some cases, to provide for future expansion of the Viaduct using details that would ease the construction process for expansion. These hinges, by design, have little or no flexural capacity, and only that shear capacity afforded by the plain concrete section. In addition, the box girder roadway included transverse expansion joints on beam seats within the roadway box girder spans, allowing for thermal expansion and contraction of sliding joints along the length of the Viaduct. Once such joints move off of their seat, the roadway girders loose vertical support.

The effect of these major differences in structural systems is that the Alaskan Way Viaduct is spared from two major weaknesses that afflicted the Cypress Viaduct – frame hinges and internal roadway girder expansion joints. Both of these details in the Cypress Viaduct contributed to the catastrophic nature of its collapse in the 1989 earthquake. It is also worth noting that while Cypress Viaduct was founded on pile foundations, there was no evidence reported that foundation failure, per se, contributed to the collapse of the Viaduct. Foundation soil conditions did, however, have an influence on the motions experienced by the superstructure, as they will at the Alaskan Way site.

4.0 Seismic Hazard Evaluation

Geotechnical earthquake engineering studies were conducted by the Committee to develop a hazard curve for the Viaduct, relating peak ground acceleration (PGA – g) to an earthquake recurrence interval. This data was then used to evaluate the relative risk of different structural options to repair the Viaduct. The repair options can be evaluated relative to earthquake probabilities that are commonly used in the design of new bridges. Thus, by selecting an acceptance criteria that is commensurate with the design philosophy of new structures, the repair options may be developed to meet the ground motions from the hazard curve that match the earthquake recurrence intervals for the applicable design events.

The ground motion hazard curve for the Viaduct site was developed using the results of regional probabilistic hazard analyses conducted by the United States Geological Survey (USGS). Specifically, equal hazard response spectra corresponding to ground surface motions for hypothetical "rock" sites were used to define input motions for subsequent use in a site specific ground response analysis of bents 97-100 of the Viaduct. The USGS uniform hazard spectra were developed for motions having recurrence intervals of 100 years, 200 years, 300 years, 500 years, 1000 years and 2500 years.

For each of the above recurrence intervals, earthquake time histories were developed to approximately match the uniform hazard spectra. Time histories were developed by modifying the north-south and east-west components of the magnitude 7.1, April 29, 1949 Puget Sound Earthquake as recorded at Olympia, Washington. The subsurface soil conditions in the vicinity of Bents 97-100 were then characterized in the analytical model as consisting of 60 feet of loose fill and Estuarine deposits overlying very dense, glacially consolidated sediments. The computer program SHAKE was then used to compute the one-dimensional response of the soils underlying the Viaduct alignment for each of two time histories at each recurrence interval.

Typical results of the site-specific ground response analysis are presented in Figure 1, which shows the response spectra at the ground surface corresponding to an event with a 200 year recurrence interval. The two spectra represent the two time histories generated for this recurrence interval.

The hazard curve for ground motion presented in Figure 2 was developed from the results of the site specific ground response studies. The hazard curve contains approximations of the hazard data for peak ground accelerations at the ground surface. Two additional hazard curves are shown for spectral accelerations (structural response curves) corresponding to structural periods of 1.5 and 2.0 seconds.

4.1 Nisqually Earthquake

The February 28, 2001, Nisqually Earthquake was widely recorded throughout the Puget Sound region. Fortunately, there were three recording stations within about 1000 feet of the Viaduct. The locations of these stations, along with the peak ground accelerations recorded at each station, are shown on Figure 3. The response spectra for these sites are shown on Figure 4, along with a standard AASHTO spectrum. As indicated on Figure 3, the three stations show an average EW recorded peak ground acceleration of about 0.19g. As reflected on the hazard curve in Figure 2, the 0.19g ground acceleration at the site would have a recurrence interval of about 150 years.

4.2 Liquefaction

Prior studies conducted by WSDOT (WSDOT, 1995-6) indicated that the Viaduct alignment is underlain by loose fill soils with a median Standard Penetration Test (SPT) blow count of about 10. Cohesionless soils with this magnitude of SPT resistance typically have a high susceptibility to liquefaction. The researchers who completed the WSDOT study concluded that widespread liquefaction would occur along the Viaduct for a peak ground acceleration of

0.16g. The researchers also indicated that widespread liquefaction would also likely result in failure of the Alaskan Way Seawall, and massive lateral spreading of the adjacent soil. Such conditions would likely result in substantial ground displacements at the Viaduct (i.e. displacements in excess of 1 foot) that could result in the collapse of adjacent sections of the Viaduct.

Numerous liquefaction features were observed following the Nisqually Earthquake. These features typically included sand boils (small volcano-like features), ground settlement, and the intrusion of sand and water into basements. Most of these features were observed in the South of Downtown district and the Harbor Island area of Seattle. However, none of these surface manifestations was observed beneath the Viaduct. Furthermore, there was no indication of distress or failure of the Alaskan Way Seawall following the Nisqually Earthquake.

Thus, while conventional methods of computing the occurrence of liquefaction indicate that the soils along the Viaduct should have liquefied during the Nisqually event (0.19g PGA), the actual performance of the soils in this area did not experience major liquefaction. Accordingly, for the purpose of developing recommendations for the repair of the Viaduct, we suggest using a threshold acceleration of approximately 0.25g to represent the level of earthquake ground shaking that might be required to trigger a failure of the Alaskan Way Seawall and associated massive lateral spreading that could jeopardize the integrity of the Viaduct. As indicated on the hazard curve presented in Figure 2, the 0.25g acceleration would have a recurrence interval of about 350 years. This level of ground shaking would have a 10% probability of occurrence in a time span of about 35 years.

5.0 Technical Reports and References

The Committee collected available data and technical resources in order to understand both the original design and current condition of the Viaduct structure. The scope of review encompassed the available design plans, recent repair plans, archived construction documentation, and a series of engineering reports on the Viaduct structure and foundation conditions. The major items are listed below, with annotations reflecting their application for this report.

5.1 Plans – (described "as-built")

Contract 3738 Station 34+02.00 to 56+53.97 (North abutment to Bent 53)

Contract 3952 Station 56+53.97 to 97+49.50 (Bent 53 to 115)

Contract 4104 Station 97+49.50 to 101+55.50 (Bent 115 to 121)

Contract 5081 Station 101+55.50 to 111+13.50 (Bent 121 to 136)

Contract 5262 Station 111+13.50 to 141+04.66 (Bent 136 to South abutment)

Contract 0706 Repair to Fire Damage Bents 105-108

Earthquake Repair, Phase I - These plans document the emergency repairs undertaken in March and April 2001 immediately following the earthquake. This work has been completed.

Earthquake Repair, Phase 2 – These plans are for the completion of repairs to Bents 97-100. This work has not yet been completed.

5.2 University of Washington Seismic Vulnerability Studies

Geotechnical Engineering Aspects, July 1995 (Ref 1) – This report summarized available and collected information on the subsurface conditions and foundation types along the Viaduct, provided a probabilistic seismic hazard analysis and an estimate of site specific ground motion parameters for the design level earthquake, investigated liquefaction potential of the soils along the site, and estimated foundation stiffness parameters for one typical foundation.

The report concluded that the viaduct may be susceptible to earthquake induced vertical settlements from a bearing capacity failure of the timber piles in the WSDOT section that were driven into the underlying glacial till. The report also concluded that the viaduct would be subject to ground displacements of 3 to 4 feet during the design earthquake (10% chance in 50 years) and that these displacements would be substantially greater with a failure of the Alaskan Way Seawall. Finally the report concluded that liquefaction would be widespread resulting from earthquake ground motions having a 50% probability of occurrence in 50 years (i.e. recurrence interval of about 70 years). Actual performance of the Viaduct during the Nisqually event indicates that potential catastrophic performance of the viaduct as related to liquefaction has a higher threshold than that suggested in the WSDOT report (Ref 4).

WSDOT Typical Unit, June1995 (Ref 2) – This report analyzes one "typical" 184' long unit of the southern section. The analyzed frame is a straight, symmetrical, three-span (56.83,70.33,56.83 ft) double deck unit. This unit represents the most common construction type. The frame was analyzed for both fixed base and pinned base conditions but foundation flexibility was not included. Structure natural periods were found to be 0.8 and 0.9 seconds in the transverse and longitudinal directions respectively for the fixed base condition and 1.6 and 2.0 seconds for the pinned base condition. (Analysis of Bents 97-100 by TY Lin International indicates that inclusion of foundation flexibility increases the fundamental period of the rigid base condition to approximately 1.5 seconds.)

Both linear response spectrum analyses and non-linear ultimate displacement (pushover) analyses were conducted. Seismic vulnerability was characterized in accordance with ATC 6-2 capacity/demand ratios. The non-linear ultimate displacement analysis was also examined by methods advocated by Professor M.J.N. Priestley, UCSD.

The analysis considered only the design level earthquake (500 year return period). Site specific ground motions were used as well as typical code level spectra.

Several structural deficiencies were noted including column splices at the top of the footings, footing shear, column/girder joint shear, and development of reinforcing steel.

The study did not investigate irregular units such as those with out-rigger columns or transitions in width. The irregular units can be expected to exhibit torsional response in addition to the transverse and longitudinal response. It can be expected that such units would show similar or worse deficiencies than the regular units investigated.

SED Typical Unit, July 1995 (Ref 3) – This report analyzed a similar unit to that described above for the northern section of the Viaduct. Analyses, methods, results and limitations were similar to those described above for the WSDOT Typical Unit.

Summary Report July 1995 – This report summarizes the two structural vulnerability reports described above.

5.3 Data furnished by WSDOT Bridge Preservation Office on CD May 10:

No comments are offered on this material.

February 28 and March 2, 2001 RABIT Inspection. This is the initial post earthquake inspection documentation, followed up by four photographs of Pier 100 on March 2, 2001.

March 20-22, 2001 Cursory Inspection. This inspection was the response to the initial RABIT inspection and site visits by Bridge Preservation Office personnel.

March 28, 2001 Expansion Joint Inspection. This inspection was performed in response to a deck joint angle iron that fell to the street below the Viaduct. Loose expansion joints were found at 27 expansion bents with loose pieces from 1' to 12' long.

March 31, 2001 Inspection (there are only photos for documentation). This inspection was conducted as part of a maintenance operation to remove loose concrete on the bridge. As part of this operation, additional distress at Piers 97, 98 and 100 were discovered, initiating the emergency repair work and further inspections that followed.

April 2, 2001 Inspection (there are only photos for documentation). This inspection was conducted from Pier 97 through Pier 103 in response to the findings on March 31, 2001. During this inspection, crack monitors were installed at various locations on the bridge between Piers 97 and 100. Selected photos from this inspection are tabulated in the April 14-29, 2001 inspection. The remaining photos are tabulated with the crack monitoring information.

April 5, 2001 Post Tensioning and Shoring Photographs.

April 13, 2001 Pier 97 Post Tensioning Photographs.

April 14-29, 2001 Inspection. This inspection was initiated to provide comprehensive data regarding the condition of the columns from Piers 53 through 183, in response to the inspection findings of March 31 and April 2, 2001.

Monitoring of Structure in Vicinity of Pier 100. This monitoring, which is on going, has two components. The first component consists of inspecting crack gages installed across selected cracks in the bridge columns, edge beams, and floor beams at regular intervals. The second component consists of surveys of established reference points on the columns at regular intervals. In both cases, structural movements between monitoring intervals can be found and tracked. From April 3 through May 10, 2001, there was no recorded movement of the structure, based on 7 observations of the crack gages and 4 observations of the survey reference points.

Tabulation of Bridge Construction Contracts and Station Points. This is a tabulation of the station of each bent and cross street and the contract under which it was built.

Biennial Bridge Inspection Reports. All the inspection reports on file (1956-2001) are provided, although associated photographs are not included.

Pile Driving Records. Records for piles in the vicinity of Pier 100 and from test pile driving records for Piers 136 through 157 were included.

Special Provisions. Contract 3935 and Contract 5262 were reviewed. Contracts 3738, 4104, and 5081 were not included.

5.4 Geotechnical Report, Alaskan Way Viaduct, Phase III Seismic Vulnerability Study, June 1996 (Ref 4).

This report further investigated the seismic vulnerability of the seawall and risk of liquefaction south of Station 125+00 identified by the UW Geotechnical Report. It concluded that a significant risk of failure of the seawall and of area-wide liquefaction existed in the design level (500 year return period) earthquake, since the computed liquefaction threshold was about .16g. Mitigation measures (jet grouting) were recommended for two areas behind the seawall and structure replacement south of Station 125+00 was recommended. Cost estimates for the jet grouting behind the seawall were provided.

5.5 Phase III-B Preliminary Seismic Retrofit Plan and Cost Estimate, July 1996 (Ref 5).

This report provided a recommended plan for retrofit and an estimate of cost to construct the retrofit. Costs for demolition of the existing structure and a replacement in kind were also provided. This study estimated that costs (in 1996 dollars) for structures only would be:

Demolition of existing structure	$119 million
Retrofit	$344 million
Replacement in kind	$531 million

Retrofit measures included all those identified in the WSDOT studies plus some retrofit measures for approach ramps. Foundation retrofit and utility relocation costs were included.

5.6 Bridge Load Rating Study, CES Engineering, June 2001.

This report was in progress at the time of this review. A briefing on the results to date was provided to the Committee on 17 May and updates to results have been furnished through 29 May. The results of this work to date indicate that flexural capacity of the structure is adequate for normal loadings but that there are areas of the structure for which shear ratings are deficient. The rating study was conducted on the basis of structure condition as reported in the latest biennial inspection report (December 2000). Damage from the 28 February earthquake was not considered in the analysis.

Committee Commentary on Bridge Load Rating and Condition

Corrosion - Corrosion of the reinforcing bars has been noted in the inspection and is visible at several locations in the structure. It is unlikely that significant loss of section has occurred in the main longitudinal steel but there may be some loss of section in the stirrups and ties. It is the Committee's opinion that one of the chief causes of the loose concrete observed during the post-earthquake inspection is corrosion in the reinforcing. Cracking in the structure provides access to the reinforcing for salt laden air, oxygen, and water. This means that active corrosion can be expected to be in progress at these locations. The age of the structure and probable permeability of the concrete are such that the passivity of the concrete is no longer a protection for the reinforcing steel. It can be expected that corrosion problems will increase in the future. Corrosion products on the reinforcing steel create tensile stresses in the concrete cover. These tensile stresses reduce the ability of the cover to provide development of the longitudinal reinforcing even if separation of the cover or loss of section has not yet occurred. The rate of corrosion of the reinforcing steel is a function of the permeability of the concrete, extent of cracking, application of deicing salts (not a factor in this structure) and salt content of the atmosphere. It is unlikely that corrosion alone would cause damage requiring replacement for structural reasons within the next 15 years despite the apparent corrosion observed. This opinion is based on observation of other structures in the area of similar or older age. (Spokane Street Viaduct, Ballard Bridge, University Bridge, Magnolia Bridge for example).

Foundations - The major indication of distress due to foundation performance is the settlement of the east footings at Bents 98 and 99. The settlement is approximately 0.4 feet at Bent 98 and 0.5 feet at Bent 99 based on comparison of measured top deck elevations with plan elevations. It is unlikely that this settlement was caused by the recent or previous earthquake events. If this settlement were induced by earthquake, the magnitude of the resulting moments would have caused major yielding of the moment reinforcing of the eastern edge beams at Bents 97-100. Such damage is not evident. This settlement could have been associated with construction.

Load ratings - Load rating analyses performed by CES based on the pre-earthquake condition indicate that the rating for the structure is satisfactory except for some areas of deficiency (rating factor < 1.0) in shear in the cross beams and straddle bents. Cracks have been evident in some of the cross beams for several years (since 1960) based on the previous inspection reports. This cracking is common and not necessarily reason to restrict traffic.

Engineering Issues with Inspection and Rating - Design code procedures for shear in reinforced concrete have changed significantly since this structure was designed. It is therefore common for older structures to indicate rating deficiencies in shear. The most current design specifications (AASHTO LFRD 2nd Edition, 1996) would suggest that a strut and tie analysis be done in regions near the introduction of concentrated loads or supports. It is the understanding of the Committee that WSDOT would not post a structure rating as deficient in shear without more detailed investigation.

It should be noted that load ratings assume simultaneous occupancy of lanes by fully loaded design level trucks, or by permit level loads with concurrent occupancy of other lanes by normal design level loads. The design live loads used in the AASHTO Code represent the maximum loading effect expected on the structure. Loads that occur frequently are less than the full design live loads. The more common loads are reflected in the use of a 25% reduction in truck loading when checking for fatigue.

The typical load rating analysis also does not account for fatigue damage that may be present at the welded splices of the upper corner reinforcing bars. At least one of these bars is known to have failed (at Bent 100N on the east side). This represents a 33% loss of capacity at this particular section. This welded detail is one that would not be allowed under the current codes, both due to the type of weld and due to the reinforcing and weld metal material used in the 1950's. Since this detail is typical throughout the northern section of the Viaduct, there is a reasonable probability that other bar splice details have fractured elsewhere along the Viaduct.

Programmed life expectancy was changed by decree in approximately 1963 according to the inspection reports. Prior to 1963, the structure had a programmed replacement date of 2002, after 1963 this was changed to 2027. Deck cracking and wear are evident from the inspection performed after the earthquake although the area of patching was found to be less than 1% of the total area. There are numerous areas of exposed reinforcing steel on the deck and on edge beams, as well as signs of deck wear. The inspection for corrosion was apparently limited to the deck. Testing of samples taken from the deck indicated 33% (of samples taken) had chloride contents in excess of 2 pounds per cubic yard. This is above the threshold limit for corrosion (1.4 lbs/cy). Maintenance costs can be expected to increase as corrosion progresses. There is no evidence to justify extending the replacement date beyond 2027, and insufficient evidence to significantly decrease the expected remaining life below 15 years. It is recommended that a thorough examination of the extent of corrosion be undertaken if the structure is expected to remain in service beyond 2015.

5.7 Construction Records from Archives

Resident Engineer's final reports

Contract 3935

Contract 5262

These reports are the Resident Engineer's final report of construction detailing contract changes and construction progress. No significant contract changes were noted. It was noted that substitution of cast-in-place concrete piles and composite timber-concrete spliced piles was allowed in lieu of steel H piles in many footings due to delays in steel delivery. The exact location of the substituted piles was not shown.

Indicator pile driving records for Contract 5262. These records are the blow counts for each foot of driving the 25 timber test piles required through the length of this contract.

5.8 Bridge Deck Condition Inspection Report, May 2001

This report details the findings of an inspection of the condition of the bridge deck made while the bridge was closed for emergency repairs following the 28 February earthquake. Findings included:

Roadway	Test	Samples	Results	Criteria or comment
NB (Upper)	Delamination	9/86	1895 sf	0.4% of total area
	Chloride	7/86	35%	> 2 lb/cy
	Cover	9/86	19%	< 1 inch
SB (Lower)	Delamination	4/86	1363 sf	0.4% of total area
	Chloride	4/86	26%	> 2 lb/cy
	Cover	4/86	16%	< 1 inch

Sufficiency ratings were estimated to be 66.17% for the upper deck and 54.88% for the lower deck. A protective overlay was not recommended.

5.9 Alaskan Way Viaduct Scoping Study, Draft Summary, Office of Urban Mobility, May 2001

This draft summary of potential improvement concepts was presented at the 16 May meeting in Lacey. This study (in progress) will consider all feasible options for transportation improvements in the SR99 corridor including replacement in kind, a 6 lane single level structure, cut and cover tunnel, surface roadway improvements, a bored expressway tunnel, combinations of structure and tunnels, and light rail/monorail options. Little data relevant to the Committee's study is yet available.

5.10 Rating Study of Frame Unit 97-100 in Damaged Condition, TY Lin International, May 2001 (Ref 6)

This study examined the rating factor to be assigned to the damaged frame Bents 97-100 in the condition after the February 28 earthquake. The study included non-linear ultimate displacement analysis of the entire frame. Dead and live loads were applied to the frame in the condition found after the earthquake including fractured reinforcing bars and residual lateral deflections. Flexural strengths were found to be satisfactory but shear ratings for the crossbeams were deficient. This finding is similar to the results of the CES rating study described above.

6.0 Evaluation Criteria

6.1 Merits/Explanation of Retrofit vs. Replacement

The existing structure will not last forever. Replacement will be necessary sometime. The real question is when the present Viaduct structure should be replaced, from the standpoints of both safety and economy. The replacement might be soon, in order to minimize seismic risk by minimizing the time of exposure of the present structure. Alternatively, the replacement might be deferred to the time when the structure reaches the end of its normal serviceable and functional usefulness. In the latter case, some retrofitting would be necessary to reduce the seismic risk to an acceptable level.

6.2 Replacement and Retrofit Issues

6.2.1 Life Cycle

The Viaduct is now approximately 50 years old. Bridges are commonly assumed to have a useful service life of 75 years. This estimate of useful lifetime is based on gradual deterioration and functional obsolescence, without regard to the possibility of seismic events reducing the life expectancy. The Viaduct is now perhaps two-thirds of the way through its useful life cycle. This fact weighs on the decision of whether to retrofit or replace. Retrofitting can reduce the seismic hazard. Additional resources will be required to overcome the effects of deterioration, wear, corrosion, and functional obsolescence in order to extend the life of the Viaduct beyond its normal useful life.

6.2.2 Base Isolation Retrofit

The concept of base isolation was not addressed in prior studies of the Viaduct. Base isolation is a means of lengthening the period of the structure so as to lower the spectral response to earthquake ground motion. In the case of the Viaduct, this approach would require a separate base isolation device at the top of each footing in the Viaduct. There are several reasons not to advance this approach. First and foremost are the aforementioned issues with structural condition and useful life of the Viaduct. Second is the variable foundation condition along the length of Viaduct, including soft, long period soils that do not benefit fully from base isolation. Lastly, the logistics and maintenance of a subterranean mechanical system are problematic for this Viaduct structure.

6.2.3 Reliability of Replacement vs. Rehab

The Committee believes that a replacement structure, built using the best of today's technology, could have a serviceable lifetime of 100 years, although functional obsolescence could shorten the useful lifetime to less than 100 years. Changing codes, particularly seismic codes, can also alter the recognition of utility and safety of a structure. This certainly has happened to the existing structure. Codes are changed as a result of new evidence of unsatisfactory behavior, either in actual earthquakes or in laboratory testing. Codes will change in the future, but it is unlikely that we will see the extensive change in seismic codes in the next 50 years that has occurred in the last 50 years. The Committee is using a 75 year life span for this study, which is consistent with new codes and standards.

A retrofit/rehab would be less reliable than a replacement. How much less is difficult to quantify, but the Committee believes that a retrofit would be significantly less reliable than a replacement structure because of the deterioration that has occurred with age and due to the different design standards for new construction. The ductility designed in to a new structure will result in damage and perhaps temporary closure of a new structure, but not structural collapse at the design event. In contrast, due to the lack of ductile reinforcing details, the existing Viaduct can be expected to suffer more catastrophic and sudden collapse, typical of a brittle structure. Thus, there is even greater distinction between a new structure and the existing Viaduct than is indicated in the exceedance probabilities alone.

6.2.4 Other Issues

1. A replacement structure would offer an opportunity to address some of the functional deficiencies of the present structure, even within the bounds of a "replacement in kind." Some of these are:
   a. The hazardous point where the northbound Western Avenue onramp joins Highway 99 on a blind curve while entering the Battery Street tunnel.
   b. The merge where the traffic from the Columbia Street onramp joins the fast-moving traffic.
   c. The congestion caused by lack of good on/off access to the new sports complexes.
2. Long-Term Value

   A new structure will have greater long-term value than a retrofitted 50-year old structure. However, if the replacement were deferred 20 years, at that time it would have greater long-term value than a structure replaced now, which would be partway through its life cycle at 20 years.
3. Retrofit Same as Replacement

   It has been reported that in California structures have been retrofitted even though the cost is the same as, or higher than, replacement. The reason is that although it is more difficult and costly, traffic can be maintained during the retrofit. Therefore, one constraint on replacement for seismic safety is the broader transportation issue of traffic maintenance.

   For the Viaduct, a full retrofit bringing the seismic resistance up to modern standards might proceed as follows:
   a. Shore and brace the existing roadways with braced or rigid-frame temporary steel supports.
   b. Remove existing columns and longitudinal beams. Encapsulate footings in new drilled-pile supported footings.
   c. Cast new columns and longitudinal beams, with provisions for anchoring supplementary post-tensioning to upgrade existing cross beams in shear and flexure. This may involve demolishing a portion of the ends of the cross beams, so that proper strength and confinement may be provided in a potential hinging zone.
   d. Recast columns and longitudinal beams.
   e. Remove and reuse temporary steel shoring.

6.2.5 Build New In-Kind

The Committee realizes that when the Viaduct is replaced, it will not be with one of the same capacity and configuration. It is beyond the scope of this report to evaluate these expanded alternatives. In order to make a comparison among equals, the Committee assumed that the replacement structure considered for comparison would have a configuration and location similar to the existing structure. Based on our review of the plans, the roadway area of the basic Viaduct and ramps is approximately 920,000 square feet. WSDOT has used a total area of 1,200,000 square feet in their estimates of replacement costs. The Committee has used the latter figure in preparation of its cost estimates.

A very important consideration in replacement of the existing structure is how traffic can be maintained during the six to eight years required for demolition and new construction. The cost to the 100,000 daily users of the Viaduct, and the cost to Seattle in general, will probably make total closure to traffic an unacceptable option. Locating the replacement corridor away from the existing structure could solve this problem. However, there are also options for replacement on the same alignment that can be explored.

A possible solution for reconstruction on the same alignment is to use a steel detour bridge. This temporary structure is used to route traffic around the construction site. The new construction could be done in segments of the total length, with a temporary double-deck steel structure located over the existing Alaskan Way surface street. The temporary structure would be of bolted steel construction, with a configuration west of the Viaduct similar to the existing WSDOT section. New pile foundations would be required, located adjacent to the existing pile caps. Figure 5 shows how the transition between the detour bridge and the existing structure might be accomplished. The temporary structure could be dismantled and relocated as the reconstruction progresses along the 2.1-mile length of the Viaduct. This is a difficult, but necessary task, and the Committee has included rough cost estimates for providing a temporary bypass during construction of a new Viaduct.

7.0 WSDOT Repairs to Unit 97-100

Damage to the Structure

After the February 28 Nisqually earthquake, WSDOT discovered extreme distress in the Viaduct unit that turns the curve at Washington Street. This unit showed signs of residual lateral displacement that predated the earthquake, but now showed signs of moving further to the east after the Nisqually event. Most significant was the major concrete cracking in the knee joints on the east side of the Viaduct at bents 97 and 100, along with major cracking in the transverse floorbeam at bent 100. The knee joint cracking was in the vicinity of welded reinforcing bar laps shown on the original design plans. Removal of the cracked and spalled concrete at bent 100 revealed fractured reinforcing bar at the upper knee joint where the welds terminated on the rectangular reinforcing bars.

The unit is in the part of the Viaduct that was designed by the City of Seattle Engineering Department. This unit is approximately 222 feet long. It consists of four transverse frames linked together by haunched girders that frame into cross beams that are part of the transverse frames. Exterior edge girders span longitudinally between the transverse frames, however these edge beams are lightly reinforced, and were designed primarily as longitudinal framing members, with little contribution to live load capacity.

Bents 97-100 are largely independent of other structural units. The end bents share a common footing with the end bents of adjacent units—the 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 curve, whereas the bulk of the Alaskan Way Viaduct is straight. The lower deck of this unit also contains a widening from 40' to 52' to accommodate the Columbia Street on-ramp merge lane. The curvature and irregularity of this unit may have contributed to the greater damage to this unit.

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.

An evaluation of the damaged unit was conducted using a non-linear frame analysis that included the residual lateral displacement of the unit, along with the member damage noted in field reports. The latter included modeling the damaged knee joints, cracking in the transverse floorbeam, and cracking in the longitudinal edge beams. An AASHTO HS-20 live load (the design load) was applied to the deformed structure in its damaged condition to ascertain whether the observed damage seriously limited the capacity of the unit to support traffic loads. The results of this analysis (Ref 6) indicated that the transverse floorbeams were adversely affected by the damage, and that rating of the bridge needed to be limited until repairs were made. Further analysis by WSDOT is underway to confirm that the "Phase 2" repairs planned will be sufficient in order to remove the current traffic restrictions placed on the Viaduct.

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 is planned, which includes removing the diagonal braces and replacing them with an alternative strengthening method. While this work is currently underway, it appears that a repair of the upper knee joints at bents 97 and 100 through the use of drilled in reinforcing bars may restore adequate lateral resistance to the unit.

The careful inspection of the Viaduct carried out after the Nisqually earthquake revealed a number of stations along the Viaduct where the eastern gutter line fell well below the original plan elevations. The largest of these disparities was at bents 98 and 99, where data showed a difference of 5 and 6 inches for the east gutter line. The initial judgment was that this grade data was evidence of a sudden settlement, which would have required either pile plunging or footing failure. An analysis of the unit between bents 97 and 100 indicated that such a settlement would have caused major structural damage to the edge beams – damage that is not evident in the structure. An inspection of the footings of Bents 98 and 99 is needed in order to assess this issue. A finding of local foundation failures in this region would alter our conclusions regarding the operating conditions for the Viaduct.

The damaged unit 97-100 was also evaluated using a non-linear ultimate displacement analysis as a tool for assessing lateral capacity. The resulting analysis showed that the strength limit for the Viaduct is surpassed at a structural response, or spectral acceleration, of approximately 0.26g, a value similar to that established in the UW Studies. This limiting structural value is compared to the site spectral response values shown in Figure 2, using a structural period of 1.5 seconds. Figure 2 shows that the ground motion required to cause that spectral acceleration has a return period of 210 years. This value is used as a basis for recommendations.

8.0 Retrofit Evaluations

The option of seismic retrofit has been raised through prior investigations and reports, and has been offered as one of the approaches to addressing seismic risks with the Viaduct. The Committee has reviewed the issue of retrofit from a variety of perspectives, searching for options that might address the major risks most economically, and considering the foundation and structural retrofits addressed in the UW studies (Ref 1-3). The following approaches are a summary of this review.

8.1 Local Improvement Options

Ground improvement may be appropriate as a limited solution for the zones where potential liquefaction of the soils presents a significant risk of collapse of the Viaduct. There are 2 zones where liquefaction during and following a major earthquake has special consequences for the Viaduct; Sta.66+50 to 76+50 and Sta. 81+50 to 89+50. 20 bents of the Viaduct are in these zones, from approximately bent 70 to 82, and 92 to 100.

Jet grouting has been proposed in the WSDOT report (Ref 4) as the most practicable means of stabilizing the soils against liquefaction. The estimated direct cost of $52M appears reasonable. Utility relocation will also be required for which an allowance of $17M has been made, for a total of $69M.

The Committee has investigated alternative structural solutions that we believe may be more appropriate and reliable. The existing piling under the footings consists largely of unreinforced concrete piling, and is vulnerable to bending and shear under the lateral forces of liquefied soil movement. To give assurance against pile failure, it is necessary to provide both lateral restraint and vertical support to the footing. This led to the conceptual scheme in which moderate diameter piles are installed around the perimeter of the footings and the footings themselves are reinforced and thickened so as to provide full vertical support. Piling would be socketed into the till by drilling so as to develop the necessary resistance to lateral forces. Piles would be sized to keep the same stiffness as the present piling so as to not change the structural response of the Viaduct. A cost analysis is included in Appendix A. The necessary strengthening of the footings, for which the costs have been included, will also retrofit these footings for shear, which was a matter of concern in the WSDOT reports (Refs 1-3).

It must be recognized that this foundation retrofit option has limited utility. Major liquefaction would result in pervasive failure along the stationing of the seawall. However, the return period for the event that results in bridge structure failure is less than the return period for the event expected to result in major movement or failure of the seawall (210 years, vs 350 years). Therefore, the utility of the foundation-only retrofit is limited to preventing major liquefaction from causing local catastrophic collapse, but this is not the critical event for the Viaduct.

The WSDOT document (Ref 5) states that the pile supports under bents 157-183 (southern section) are generally inadequate and that there is a danger of the piles plunging up to 5 feet under earthquake excitation. Our Committee has examined the available data and concludes that such “plunging” is unlikely unless substantially different geotechnical conditions exist in the locations for which there are no borings. The report recommended replacement for this southern section, since the authors believed that driving additional piles would potentially cause excessive settlement due to the vibration.

The Committee believes that a more economical solution for this particular issue would be to underpin the footings in this section by drilled piling, using a system suitable for the soft soils, which essentially involves grouting to enhance the skin friction. The pile installation is constrained by the low clearances under the deck but such problems have been successfully overcome in California seismic retrofit projects such as the eastern ramp of the San Francisco-Oakland Bay Bridge, using, in that case, the "Fundex" pile. Other similar pile types are available which do not produce vibration. The new piles would be drilled and grouted to the depths required, estimated to be –70 to –100. However, this retrofit is also of limited use, since the bridge structure may fail at a lower earthquake level than that producing liquefaction.

8.2 Superstructure Retrofits Proposed in 1995-96 Studies.

The Preliminary Seismic Retrofit Plan and Cost Estimate, prepared by WSDOT (Ref 5), dated July 23, 1996 sets forth a summary of the needed retrofits, prioritized on the basis of urgency, consequences, and costs. These were, of course, prepared before the Nisqually earthquake. In general, however, we believe that the WSDOT and University of Washington studies identified most of the structural deficiencies.

The Committee has reviewed these recommendations and the priority listings. We concluded that a piecemeal, item by item approach, is unwise. The retrofit of individual members inevitably changes their stiffness and affects the behavior of the other members. We therefore believe that any structural retrofit solution, beyond the repairs to be carried out in Phases 2 and 3 for bents 97-100, should be a complete retrofit of the structure (described in item 3 of 6.2.3), which would then permit a valid analysis and evaluation of the retrofitted structure as a whole. This conclusion is based on an assessment of the risk and reliability of the Viaduct subject to the seismic events that could trigger both a structural and a foundation failure. The pervasive nature of the structural limitations of the current design, coupled with the risk of an earthquake magnitude sufficient to fail a variety of current structural features within the Viaduct, makes partial retrofits an unproductive strategy.

9.0 Preferred 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 broadly accepted by the engineering profession as a basis for design, and is written into the current bridge specifications. Therefore, this same approach was adopted by the Committee as the basis for acceptable risk for the various options for dealing with the Viaduct.

After evaluation of risks and site conditions, the Committee agreed on three engineering options:

1. Option 1 is described as "Repair Only – Return to pre-Nisqually Earthquake condition". It consists of performing the repairs planned under the WSDOT's Phase I, II and III

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 is 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 is 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 with in 20 years, was rejected as an alternative. This approach addresses 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 does not address both the structural and the foundation risk, it is not included in the summary table of options.

10.0 Cost Evaluations and Comparisons

In estimating the cost components of the several options, the WSDOT format (Ref 5) has been followed. Costs were initially determined as the estimated Construction Contract price, based on the limited reference to other similar projects, and based on replacement or retrofit in-kind. Therefore, these cost estimates should be viewed as comparative rather than programmatic. The estimated costs of utility relocation/replacement were added. A 50% surcharge has been applied for Region costs, which we presume includes engineering design, mitigation, administrative and general costs incurred by WSDOT. All costs are estimated as of 2001, using a 4% escalation rate, compounded. Contingencies were applied at 25% for retrofit work and 15% for new work.

The estimated project Costs in 2001 for the several components were as follows:

Replacement (75 year life)	$870M
Full retrofit of existing structure (50 year life)	$720M

Detailed estimates are provided in Appendix A.

10.1 Present Value Analyses

For the purpose of comparing on an economic basis the several different alternative options, the present value method was applied. All alternatives were conceptually selected to provide an acceptable level of risk during periods prior to final replacement. Immediate replacement, within 10 years, with a structure having a 75-year lifetime, would result in the end of its useful life at 2086. For those alternatives in which the replacement is assumed delayed, credit has been given for remaining useful life beyond 2086.

Escalation has been applied to 2001 costs at the rate of 4% compounded. Interest on bonds has been assumed at 6% compounded.

Alternative I	Full replacement in 10 years (2011)	$760M
Alternative II	Full retrofit completed in 2011. Replace in 50 years	$948M
Limited Alternative III	Minimum retrofit now against only liquefaction (structure remains at risk) with full replacement in 20 years	$671M

Present Value Estimates are provided in Appendix B.

11.0 Recommendations

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 Viaduct 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. Thus, the recommendations arising from this report are inevitably based on estimates of the risks.

The 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. These are summarized as follows:

Options and Risks

Item	Decision/Option	Near Term Risk	Long Term Risk
1	Repair 97/100 only (return to pre-Nisqually EQ condition)	Same as current	Risk of exceedance = 210 yr event (1 in 5 chance of exceedance in 50 years, even without deterioration)
2	Replace as soon as financing, planning and engineering can be completed	10 years to complete (risk ~ 1 in 20 chance of exceedance in 10 years for 210 year event)	Replaced (standard risk 1 in 10 chance in 50 years)
3	Full retrofit to current standards	10 years to complete. (Risk same as item 2)	Replace within 50 years.

The standard for contemporary design is a 10% chance of exceedance in 50 years. This standard of risk was utilized to address time limits for all options, and also to rate the various options along with costs. The Committee limited inquiry to the double-decked mainline structures.

The Committee recommendations are as follows:

1. **The Alaskan Way Viaduct should be replaced in kind 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 does not recommend retrofitting the Viaduct as a long-term solution. The estimated costs of retrofit are similar to the estimated costs for replacement, and the Committee believes that replacement offers 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.

References

1. Kramer, S; Sivaneswaran,N; Tucker, K; “Seismic Vulnerability of the Alaskan Way Viaduct – Geotechnical Engineering Aspects,” Report No. WA-RD 363.2, July, 1995

2. Eberhard, M; De la Colina, J; Ryter, S; “Seismic Vulnerability of the Alaskan Way Viaduct – WSDOT Typical Unit,” Report No. WA-RD 363.1, June, 1995

3. Eberhard, M; De la Colina, J; Knaebel, P; “Seismic Vulnerability of the Alaskan Way Viaduct – SED Typical Unit,” Report No. WA-RD 363.3, July, 1995

4. Kimmerling, R. E.; Kramer, S.; “Geotechnical Report Alaskan Way Viaduct – Phase III Seismic Vulnerability Study,” Report MS-2494,

5. Kolle, G. A.; “Alaskan Way Viaduct – Phase III-B Preliminary Seismic Retrofit Plan and Cost Estimate,” July 23, 1996

6. T. Y. Lin International; “Damage Rating Analysis of Bents 97-100 of the Alaskan Way Viaduct,” May 24, 2001

7. Housner, G.; “Competing Against Time – Report to Governor George Deukmejian on the Loma Prieta Earthquake,” May, 1990

Appendix A

Estimated costs of Component Elements

2001 Basic Cost

A. Replacement cost		
1,200,000 sq ft @ $250.00/sq ft		300M
Traffic control		30M
Temporary Bypass		
Build ¼ at a time, 3 lanes, 2 decks		
Initial Section 240,000 sf @ $150		36M
Remove and replace 720,000 sf @ $70		50M
Transitions – 6 @ 3M each		18M
Subtotal		**464M**
Utility relocation		40M
50% Region Costs		252M
Contingencies 15%		114M
Project Cost in 2001		**870M**
Comparison with WSDOT 1996 estimate		530M
Escalation 1996-2001 @ 4% - 1.22		116M
Bypass structure	104M	
Utility relocation on bypass	20M	
Region costs on bypass and utilities	62M	
Contingencies on bypass and utilities @ 25%	47M	
Subtotal	233M	233M
Previous WSDOT Project Cost in 2001		**879M**

THEREFORE USED ROUNDED COST OF $870M IN 2001

Appendix A - continue

B. Full retrofit of existing structure		
I-280/101 interchange in California 1993 costs $210/sf		
Escalation @ 4% from 1993-1.37, Add 37% - $77/sf		
Apply to Alaskan Way Viaduct - $287.00/sf		344M
Utilities		40M
Region cost @ 50%		192M
Contingencies @ 25%		144M
Project Cost in 2001		**720M**

Appendix A - continue

C. Retrofit against liquefaction sta 66+50 to 89+50 and sta 81+50 to 89+50; 26 bents, 52 footings		
Cost per footing		
Break out pavement 2,500 sf @ $5.00/sf	12,500	
Sheet piles to-15 Reuse 4 times, 62,500/4 @ $0.70	10,000	
Drive 60 piles (3d). Remove 60 (2d). Total 5d @ $5,000	25,000	
Excavate to –3. 1,300 cy @ $15.00.	19,500	
Furnish 12 tubular steel pikes 20" dia x 1" thick walls Avg length 55'. 200#/ft. 132,000# @ $0.70	92,000	
Drive piles to till. Splicing 3d @ $15,000	45,000	
Drill 15' sockets in till, place rebar cage, concrete 4d @ $15,000	60,000	
Concrete in piles 60cy @ $100	6,000	
Rebar in piles at top 24,000# @ $0.60	14,400	
Prestressing bars 6,000# @ $2.00	12,000	
Drill through pedestal 160' @ $30	5,000	
Install, stress, grout 32 bars @ $200	6,400	
Reinf. Steel in beans and 2' cap over footing 30,000# @ $0.70	21,000	
Concrete in beams and cap 110sf @ $150	16,500	
Backfill	8,000	
Paving	4,000	
Total Direct Cost	358,000	
Contractor's profit @ 25%	90,000	
Subtotal	448,000	
Utilities	25,000	
Round to	480,000	
Region costs @ 50%	240,000	
Contingencies @ 25%	180,000	
2001 Project Cost	900,000	
Utilities		17M
52 footings @ $900,000—Round to		**64M**

Appendix B

Present Value Analyses

Alternate I. Full replacement completed in 10 years (2011)		
Start of construction – 2005		
Median year for expenditure of funds – 2008		
Project Cost, 2001	870M	
Escalated project cost to 2008 (multiplied by 1.32)	1,148M	
Present Value in 2001 (divided by 1.5)		**760M**

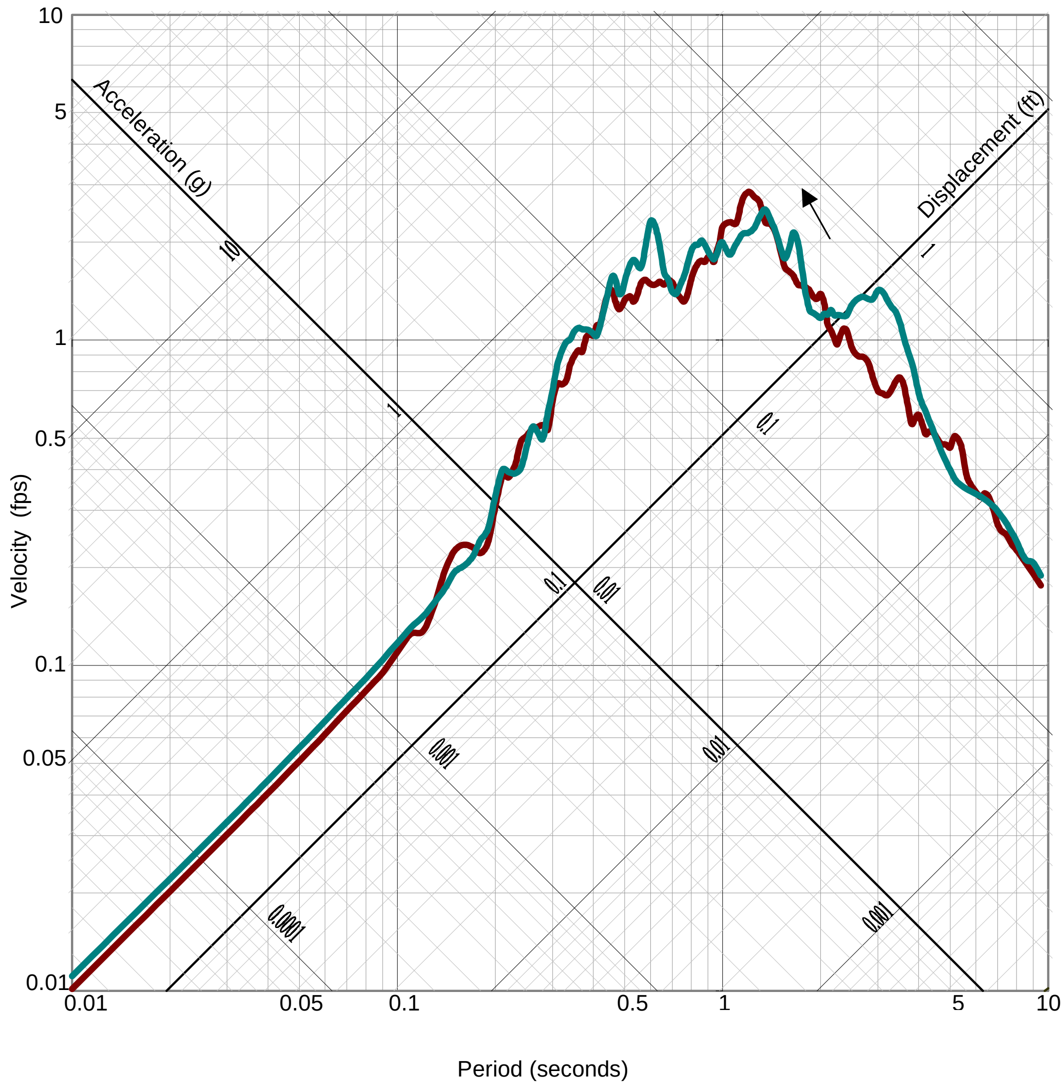
Alternate II. Full retrofit to today's standards (2001), replacement in 50 years (see Fig 6).		
Assume completion in 10 years – 2001		
Start of retrofit – 2007		
Median year of expenditure of funds – 2009		
Project cost 2001	720M	
Escalated project cost to 2009 (multiplied by 1.37)	986M	
Present Value in 2001 (divided by 1.5)		658M
Replacement in 50 years – 2051		
Median year for expenditure of funds – 2049		
Project Cost, 2001	870M	
Escalated project cost to 2049 (multiplied by 6.54)	5,690M	
Present value in 2001 (divided by 14.9)		382M
Present value in 2001 for retrofit and replacement in 50 years		1040M
However, in 2086, structure will still have a useful life of 40 years. Remaining value can be computed on a straight-line depreciation basis --$870M x 40/75 = $464M escalated to 2086 – (multiplied by 28)	12,992M	
Present value (2001) – (divided by 141)		(92M)
Net present value of Alternate III		**948M**

Appendix B – continue

Alternate III. Immediate minimum foundation retrofit against liquefaction; replacement in 20 years.		
Retrofit against liquefaction, 2001 Project Cost		64M
Replacement in 20 years – 2021		
Start of construction – 2015		
Median year for expenditure of funds – 2018		
Project Cost, 2001	870M	
Escalated project cost to 2018 (multiplied by 1.95)	1,695M	
Present value in 2001		630M
Present value in 2001 for retrofit and replacement in 20 years		694M
However, in 2086, structure will still have a useful life of 10 years. Remaining value can be computed on a straight-line depreciation basis --$870M x 10/75 = $116M, escalated to 2086 –	4,812M 3,248M	
Present value (2001) – (divided by 141)		(23M)
Net present value of Alternate II		**671M**

Appendix C – Sample Viaduct Drawings

Figure 1



1. Spectra correspond to free field, horizontal motions at the ground surface for 5% damping
2. Spectra were computed from a site-specific ground response analysis for conditions at bent 100.
3. Plots are for two different time history representations of the input motion corresponding to the USGS EHS.

PanGEO INCORPORATED	Alaskan Way Viaduct Seismic Evaluation Seatle, Washington	Response Spectra 200-Year Event Project No. 01-026 Figure No.

Figure 2

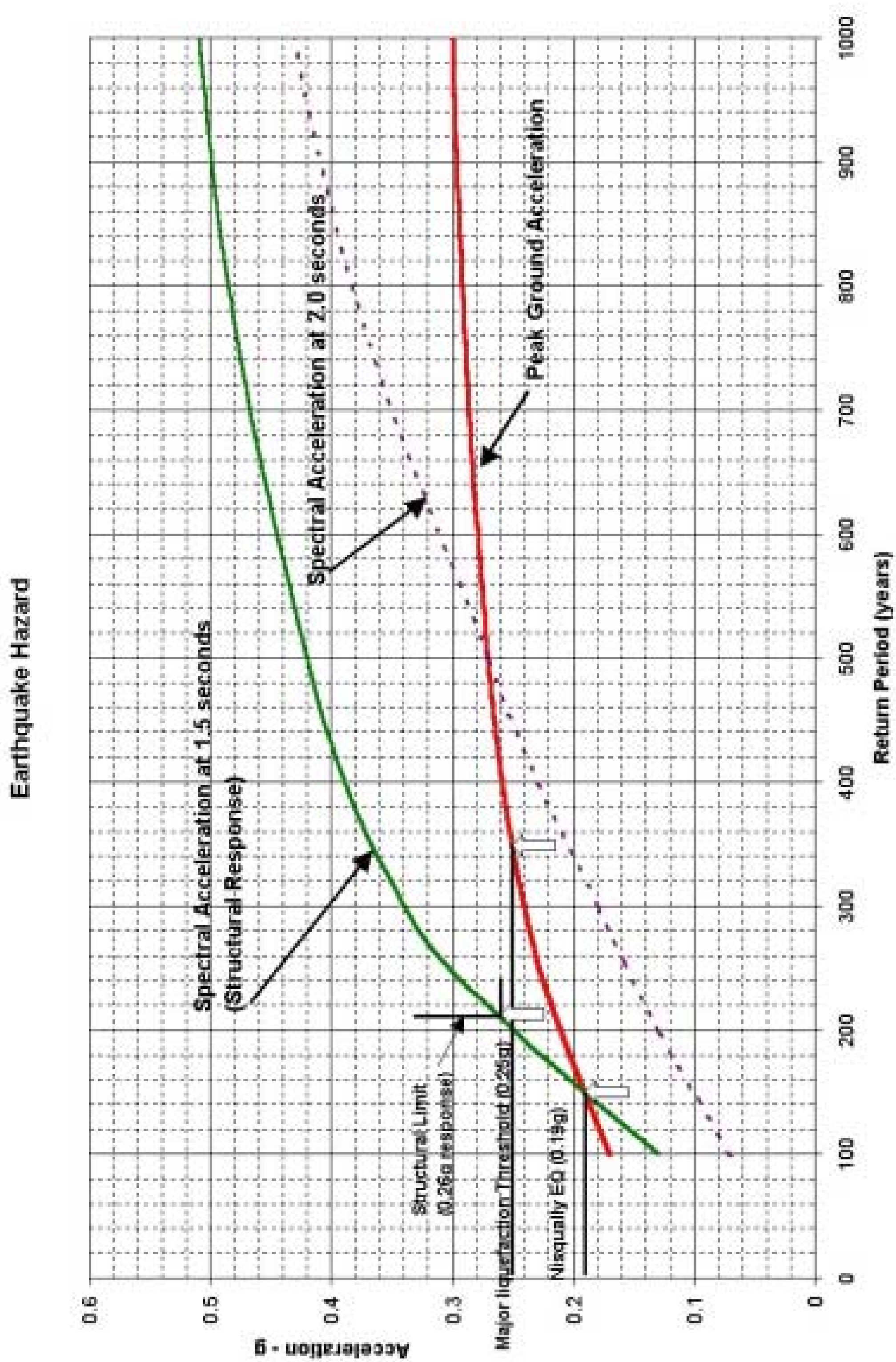

Earthquake Hazard
Spectral Acceleration at 1.5 seconds
(Structural Response)
Spectral Acceleration at 2.0 seconds
Peak Ground Acceleration
Structural Limit
(0.26g response)
Major liquefaction Threshold (0.25g)
Nisqually EQ (0.19g)
Acceleration - g
Return Period (years)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
100
200
300
400
500
600
700
800
900
1000

Figure 3

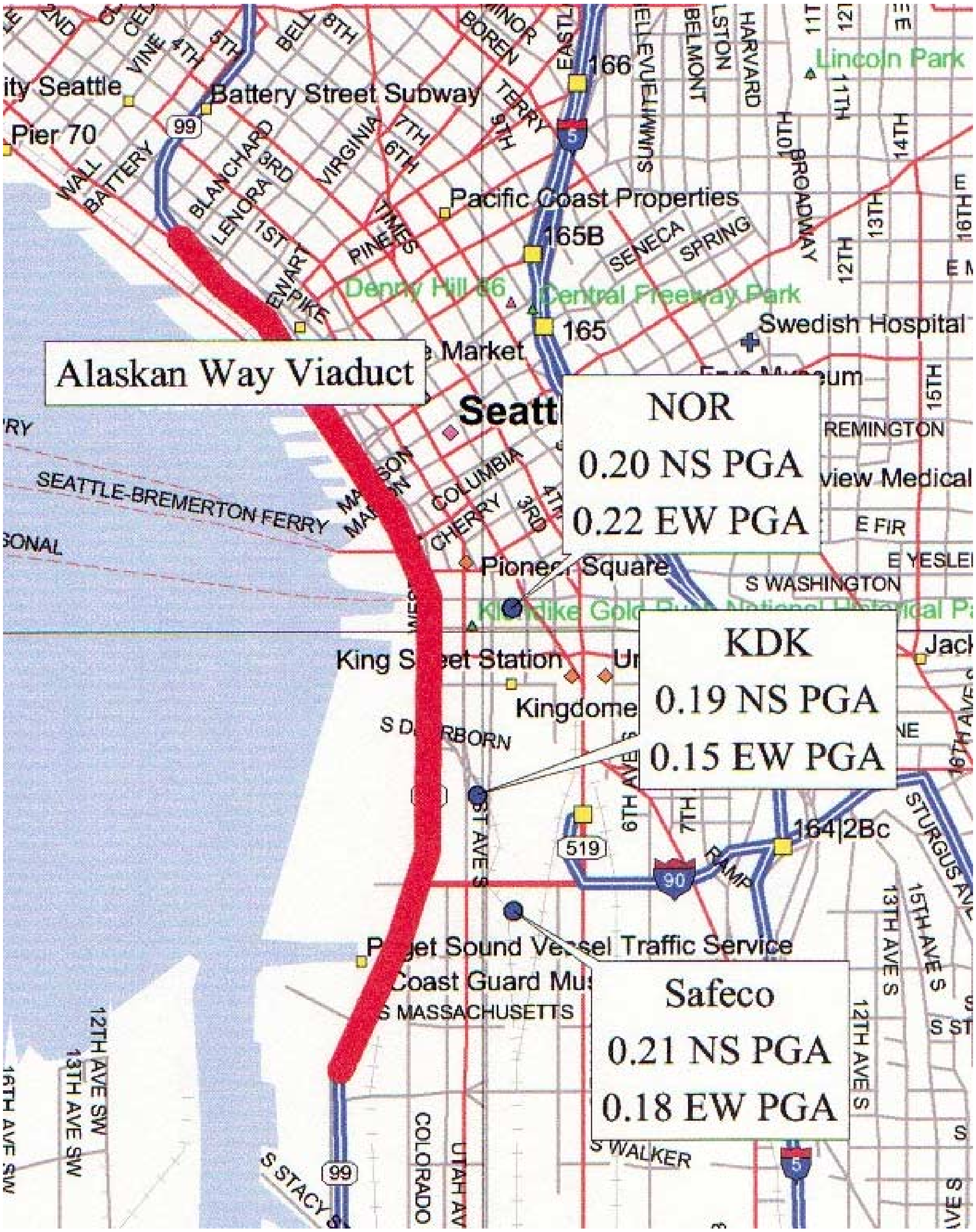
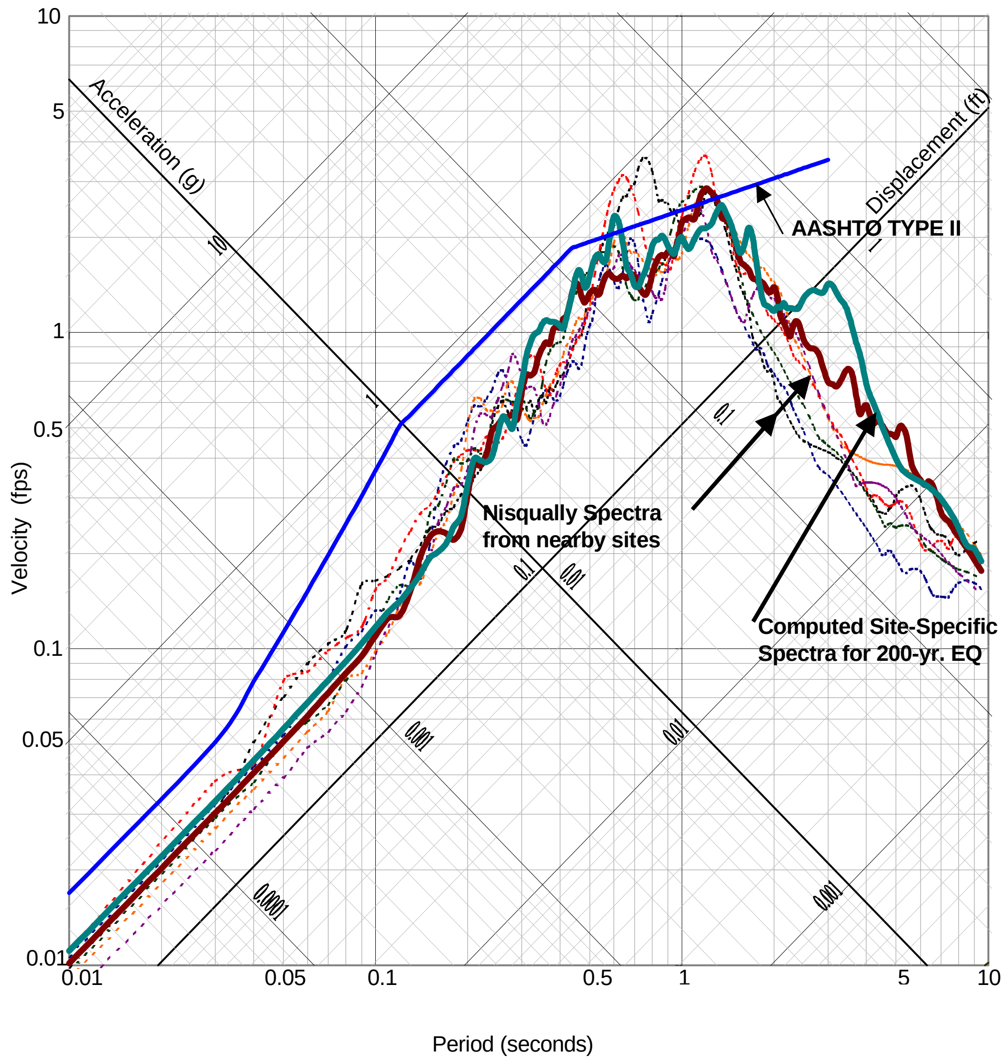

Alaskan Way Viaduct
NOR
0.20 NS PGA
0.22 EW PGA
KDK
0.19 NS PGA
0.15 EW PGA
Safeco
0.21 NS PGA
0.18 EW PGA

Figure 4



1. Spectra correspond to free field, horizontal motions at the ground surface for 5% dampir

PanGEO INCORPORATED	Alaskan Way Viaduct Seismic Evaluation Seatle, Washington	Response Spectra Comparison Project No. 01-026 Figure No.

Figure 5

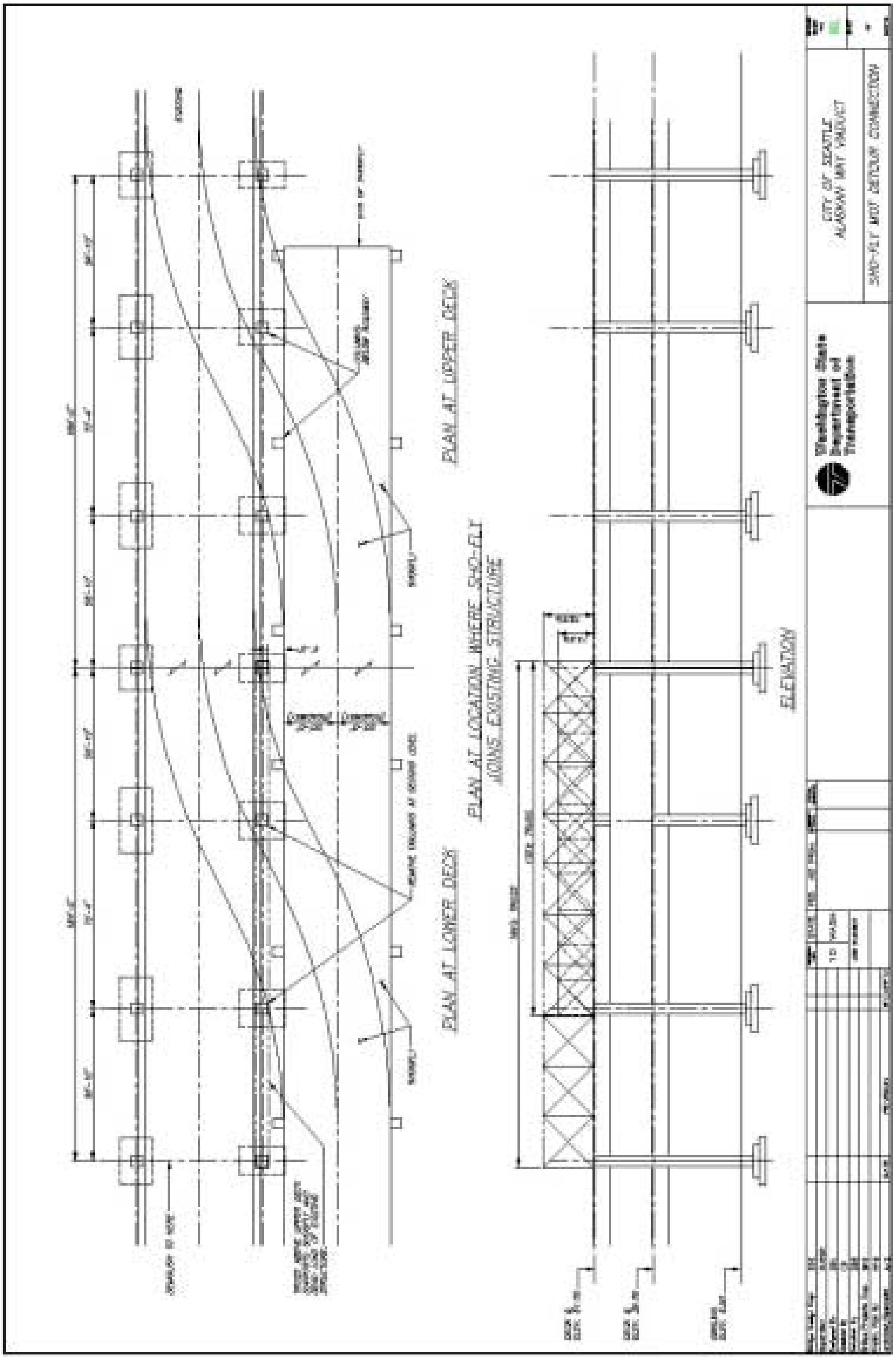
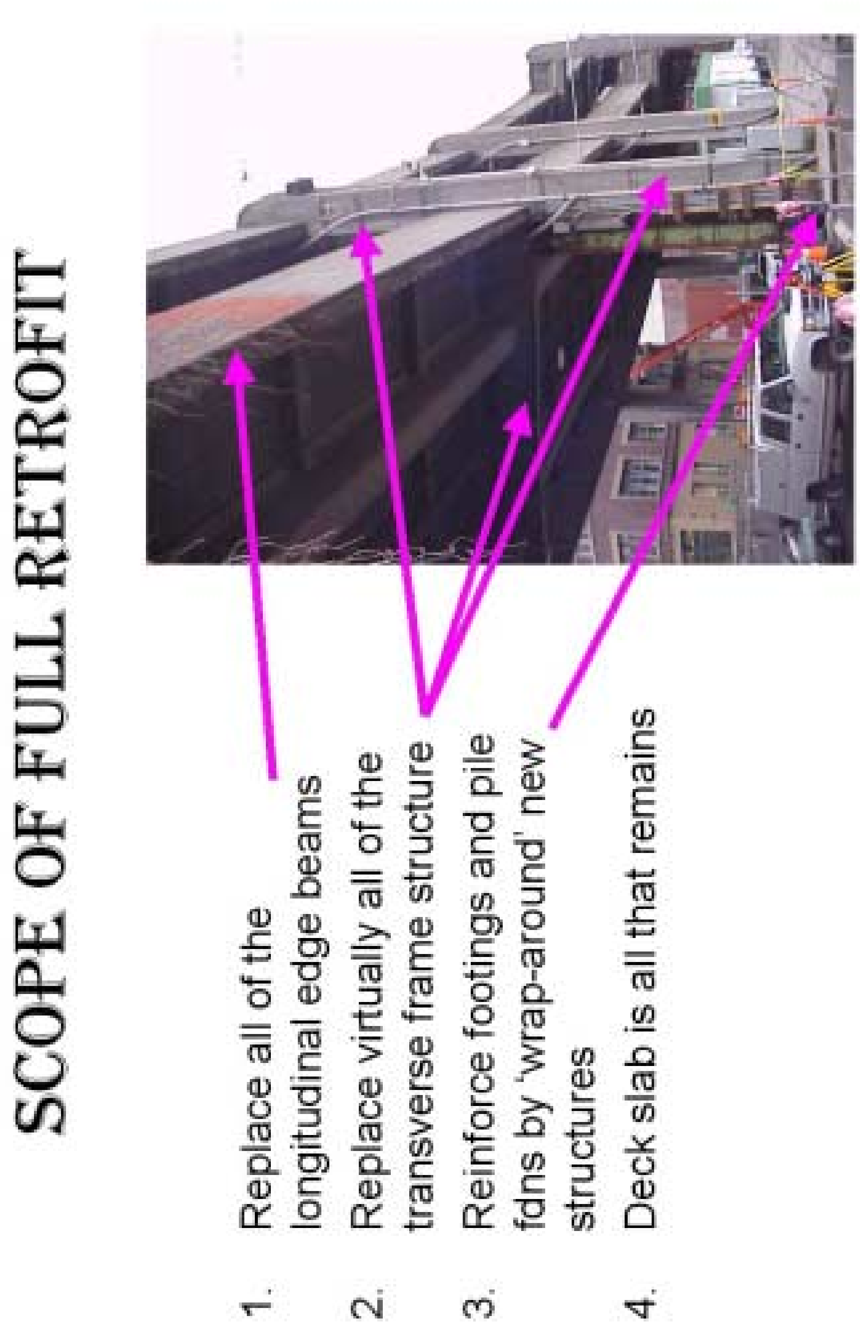

PLAN AT UPPER DECK
PLAN AT LOCATION WHERE SHO-FLY JOINS EXISTING STRUCTURE
PLAN AT LOWER DECK
ELEVATION
Washington State Department of Transportation
CITY OF SEATTLE
ALASKAN WAY VIADUCT

Figure 6


SCOPE OF FULL RETROFIT
1. Replace all of the longitudinal edge beams
2. Replace virtually all of the transverse frame structure
3. Reinforce footings and pile fdns by 'wrap-around' new structures
4. Deck slab is all that remains