Damage to Bridges during the 2001 Nisqually Earthquake

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ABSTRACT

The 2001 Nisqually earthquake, which had a moment magnitude of 6.8, damaged at least 78 bridges in western Washington State. Reports of damage sustained by bridges during this earthquake were used to correlate the likelihood of damage with the following parameters: distance to the epicenter, estimated peak ground acceleration, estimated spectral acceleration at periods of 0.3 s, 1.0 s, and 3.0 s; year built; and type of bridge. This goal was accomplished by collecting reports of bridge damage from state and local agencies, and comparing them with the population of bridges listed in the Washington State Bridge Inventory. The level of ground shaking at each bridge site was estimated from ShakeMaps, which were developed from data from the Pacific Northwest Seismic Network.

Of the four ground-motion parameters considered, the likelihood of bridge damage was best correlated with spectral acceleration at a period of 0.3 s. For a given level of spectral acceleration, bridges constructed before 1940 were the most likely to be damaged, while those constructed after 1975 were the least vulnerable. Although the number of movable bridges was small, this type of bridge was particularly vulnerable. Bridges with a steel main span were more likely to be damaged than those constructed of reinforced concrete. However, the number of steel bridges was small, and the most common type of damage to steel-span bridges was actually damage to the reinforced concrete substructure.
ACKNOWLEDGMENTS

The authors would like to thank the engineers who contributed damage reports to this study. A list of the contributors is provided in Appendix D. Harvey Coffman and his staff in the WSDOT Bridge and Structures Preservation Office, and John Buswell (City of Seattle) were particularly helpful.

Dr. Stephen Malone, William Steele, and other members of the PNSN staff provided electronic versions of the ShakeMaps, without which the analyses could not have been conducted. John Perry of the Federal Emergency Management Agency provided essential GIS support, which made it possible to link the ShakeMaps and the bridge inventory. FEMA support of the Nisqually Earthquake Clearinghouse facilitated this interdisciplinary cooperation.

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1 Introduction

The vulnerability of bridges in the Puget Sound area was investigated by analyzing reports of damage to bridges during the 2001 Nisqually earthquake. By correlating damage with bridge and ground-motion characteristics, it was hoped that the characteristics that most contributed to the damage would be identified. The characteristics that were explored included

- the year that the bridge was constructed;
- the distance from the bridge to the earthquake epicenter;
- the estimated peak ground acceleration (PGA) at the location of the bridge;
- the spectral acceleration at the site of the bridge (SA), and;
- the type of bridge that was damaged.

In the future, the observed trends could be used to prioritize post-earthquake inspections if maps of shaking intensity were available shortly after an earthquake.

1.1 Background

At 10:54:32 AM local time on February 28, 2001, the Nisqually earthquake of magnitude 6.8 occurred at location 47.1525° N, 122.7197° W. The epicenter was approximately 17.6 km northeast of Olympia, 23.7 km SW of Tacoma, and 57.5 km SW of Seattle, Washington (EERI 2001). The Nisqually earthquake occurred deep below the earth’s surface, within the subducting Juan de Fuca plate. Because of the depth of the hypocenter, approximately 52.4 km, the damage throughout the area was only moderate. Slight to moderate damage was reported to 78 bridges, with no collapses. Had the earthquake been more shallow, damage in the Olympia and Seattle regions might have been much more severe.

1.2 Research Methodology

Because the state, counties and cities keep separate records, each agency was contacted independently to obtain detailed damage descriptions and photographs of bridges that were
damaged during the Nisqually earthquake. To help with this process, a damage report form was composed to consistently extract pertinent information. A copy of the form is provided in Appendix B. Appendix D provides a list of individuals who contributed data or comments to this report.

Concurrently, the Washington State Department of Transportation (WSDOT) provided the Washington State Bridge Inventory (WSBI) in electronic form. This database provides physical and geographical information for nearly all of the bridges in the state. The WSBI was used to normalize the damaged bridge data (WSDOT 2000). The WSBI categories considered in this study were

- latitude and longitude of bridge;
- type of bridge (e.g., movable, truss, etc.);
- material used for the main span (reinforced concrete, prestressed concrete, or steel); and
- year of construction.

These data are provided in Appendix C for all of the damaged bridges. The average daily traffic data are also included in this appendix. Although this information was not used in this analysis, it could be used to analyze the economic impacts of bridge closures.

To analyze the data, each bridge had to be located, and the corresponding values for the peak ground acceleration and the spectral acceleration had to be estimated. These parameters were extracted from ShakeMaps developed by the Pacific Northwest Seismograph Network (PNSN 2001a), which are shown in Figs. 1-1 and 1-2. The PNSN, centered at the University of Washington, operates a network of seismograph stations throughout the Northwest. It is operated through a joint effort by the University of Washington, the University of Oregon, and Oregon State University, and is funded by the United States Geological Survey (USGS), the United States Department of Energy (USDOE), and the State of Washington. PNSN developed maps of earthquake intensity (ShakeMaps) by interpolating between numerous stations within the network, taking into account geologic conditions.

Access to the ShakeMap data was provided by the Federal Emergency Management Agency (FEMA), which also provided GIS support. The maps provided approximate values for the peak ground acceleration and the spectral acceleration at the location of each damaged and undamaged bridge. The map used to extract the estimated values for each bridge had a range of
48.4125° N - 46.3875° N in latitude, and 124.1125° W – 121.0875° W in longitude. Damaged bridges are identified by triangles in the figures.

Fig. 1-1: ShakeMap showing estimated peak ground acceleration (PNSN 2001b)
PNSN 0.3 s Pseudo-Acceleration Spectra (%g) Epicenter: 17.6 km NE of Olympia, WA
Wed Feb 28, 2001  10:54:00 AM PST  M 6.8  N47.15  W122.72  ID:0102281854

NOTE: These are automated maps based on instrumental response spectra, and may not be appropriate for comparison with design spectral values.

Fig. 1-2: ShakeMap showing estimated spectral acceleration at T = 0.3s (PNSN 2001b)
2 Observed Damage

The reports of bridge damage were collected from the city, county, and state governments (Appendix D). From these data, it was determined that 78 bridges had been damaged as a result of the Nisqually earthquake (Appendix C). The majority (46) of these bridges were owned and maintained by the WSDOT, and were either overpasses or underpasses along the interstate and state highway systems. The City of Seattle reported damage to 18 bridges.

2.1 Classification of Damage

The damage repair cost for each bridge was classified as slight, mild, or moderate, based on damage estimate ranges of $30,000 or less, $30,001 to $100,000, and above $100,000, respectively. The estimates provided by the individual bridge agencies are listed in Appendix C. In cases where an estimate was not provided, but where the level was obvious, the researchers categorized the damage levels themselves. According to these definitions, the number of bridges in each category is

- Slight (52 bridges)
- Mild (16 bridges)
- Moderate (10 bridges)

No damage was reported to timber or masonry bridges. The four types of bridges (categorized according to material used for main span) that were damaged were

- Reinforced concrete bridges (36)
- Prestressed concrete bridges (20)
- Steel bridges (16)
- Movable bridges (6)
The movable bridges were classified separately because of their particular vulnerabilities, e.g., lack of alignment. For the remaining 72 fixed bridges, the types of damage were classified as

- Damage to concrete (48)
- Damage to steel (6)
- Damage to beams, restrainers or joints (11)
- Settlement damage (7)

The distribution of damage type, sorted primarily by the type of bridge, is shown in Fig. 2-1. For each type of bridge, Fig. 2-1 displays the type of damage as a percentage of the total amount of damage for that type of bridge. For example, of the 36 reinforced concrete bridges that were damaged, 26 sustained damage to the reinforced concrete elements, resulting in a damage percentage of 72%.

![Fig. 2-1: Distribution of types of damage for each type of bridge](image-url)
According to the figure, concrete damage was the most prevalent type of damage for each of the three types of bridges. It had been expected that concrete damage would predominate in reinforced concrete and prestressed concrete bridges. More surprising is that damage to steel components represented only 30% of the damage to steel bridges. In comparison, 40% of damaged steel bridges were reported to have damage primarily to the reinforced concrete substructure. Most of the concrete damage to the steel bridges consisted of minor spalling of the concrete columns.

A complete list of damaged bridges, along with their physical and geographical characteristics is presented in Appendix C. Numerous photographs of bridge damage are available at http://www.ce.washington.edu/~nisqually.

2.2 Damage to Movable Bridges

Of the 78 bridges that were damaged by the earthquake, six were classified by the WSBI as movable bridges. Typical types of damage that were reported for these bridges include: damage to the leafs, dislodging of the counterweights, damage to the centerlock, and lateral shifting to the bascule towers. An example of damage to a movable bridge is shown in Fig. 2-2.

2.3 Damage due to Settlement

Significant settlement was reported for seven bridges. Six of these bridges reported settlement at the approach or within the bridge embankment. This type of damage
ranged from minor differential settlement to a reported movement of 100 yards of the approach. An example of approach settlement can be seen in Fig. 2-3. The seventh reported sighting of settlement was attributed to liquefaction around one of the piers, as shown in Fig. 2-4.

2.4 Damage to Reinforced or Prestressed Concrete

Of the 72 fixed (not movable) bridges that were damaged, 48 had damage to a concrete element. The types of damage included spalling and cracking of columns, diaphragms, and abutments. An example of concrete damage is shown in Fig. 2-5.

2.5 Damage to Steel

Only six fixed bridges sustained damage to the steel superstructure. Such damage usually consisted of bent and broken cross frames and bearing stiffeners. An example of steel damage is shown in Fig. 2-6.

2.6 Damage to Restrainers, Joints, or Bearings

Damage to the restrainers, joints, or bearings included elongated or broken restrainers, damage to movement joints, and excessive tipping of rocker bearings. Eleven of the damaged bridges sustained one of these types of damage. An example of a damaged bearing is displayed in Fig. 2-7.
3 Damage Analysis

This chapter identifies correlations between the percentage of bridges that were damaged, and the properties of the bridge and ground motion. Specifically, the analysis considered the effects of the year of construction of the bridge, the type of bridge, the distance between the bridge and the epicenter, the estimated peak ground acceleration at the location of the bridge, and the spectral acceleration at the location of the bridge. To express the outcome of these analyses in a consistent manner, the data were normalized by dividing the number of damaged bridges by the total number of bridges in the Washington State Bridge Inventory (WSBI) for each category. A total number of 8,445 bridges are listed in the WSBI. However, in each analysis, only the portion of these bridges that fell within each sorting category was used to normalize the results.

For each analysis, a series of three plots are presented. The first plot shows the total number of bridges listed in the WSBI that fit into the categories that are being analyzed. The second plot reports the number of damaged bridges in each category. The third plot shows the percentage of bridges that were damaged within each category, which corresponds to the values in the second plot divided by the values in the first plot, expressed as a percentage. The damage category “Damage to restrainers, joints, or bearings” could not be expressed in this graphical format, because there was virtually no information in the WSBI on these elements.
3.1 Effect of Year of Construction

Bridges were first sorted by the decade in which each was built. The results of this analysis for the 78 damaged bridges are shown in Fig. 3-1.

![Fig. 3-1: Effect of year of construction, separated into decades](image-url)
According to Fig. 3-1, the number of bridges constructed increased dramatically at the beginning of the 1950s, and then decreased at the beginning of the 1980s. This range of time coincides with the construction of the interstate highway system. The figure also shows that the percentage of the bridges that were damaged was largest for bridges constructed before 1940, averaging approximately 4.5%. Between 1940 and 1970, the percentage of bridges that were damaged were half that value, averaging approximately 2%. After 1970, this percentage was again reduced in half, averaging approximately 1%. Although the causes of the decline at the beginning of the 1940s are unclear, the drop at the beginning of the 1970s was expected. The San Fernando Earthquake occurred on February 9, 1971, and during the next few years, codes and practices were changed to reduce damage to structures (Moehle and Eberhard 1999).

Because of the dramatic differences between the percentage of bridges that were damaged before 1940 and after 1975, these years will serve to categorize the bridges in upcoming analyses.

3.2 Effect of Epicentral Distance

The distance of the bridge to the epicenter was the second factor considered. The distance was calculated based on the coordinates of both the bridge and the epicenter, following the procedure described in Appendix A. In this analysis, the bridges were grouped into categories that span 15 radial kilometers. The result of this analysis is displayed in Fig. 3-2.
As shown in Fig. 3-2, many bridges were damaged within the ranges of 15–30 km and 45–60 km. The range of 15–30 km corresponds to the distance to the City of Olympia, while the range of 45–60 km corresponds to the distance to the City of Seattle. As expected, the percentage of bridges that were damaged was largest near the epicenter. However, as the distance to the epicenter increased, the damage percentage did not decrease consistently. If the
intensity of the earthquake had depended only on the distance from the epicenter, the trend would have been more consistent. The correlation between damage and epicentral distance was weak, because epicentral distance does not account for the local geology. For example, the City of Seattle has a large number of bridges situated on soft soils.

3.3 Effect of Peak Ground Acceleration

To investigate the effect of the estimated peak ground acceleration (PGA), the PGA at every bridge location was estimated from ShakeMaps, as described in Section 1.2. The ground-motion characteristics for seven of the damaged bridges could not be estimated from the ShakeMap, because they were located outside of the boundaries of the map (Section 1.2). Overall, 3,312 bridges (of which 71 were damaged) were located within the range of the ShakeMap, which corresponds to an average damage percentage of 2.1%. The analysis of the percentage of bridges damaged as a function of the PGA is shown in Fig. 3-3. As shown in Fig. 3-4, there is only a weak correlation between the level of the estimated peak ground acceleration and the percentage of bridges that were damaged. From this figure, one can only conclude that bridges with peak ground accelerations above 0.2g were more likely to be damaged than bridges subjected to lower peak accelerations.
Fig. 3-3: Effect of peak ground acceleration
3.4 Effect of Spectral Acceleration

The correlation between spectral acceleration and damage was also investigated. The PNSN ShakeMap provided data for the spectral acceleration at periods of 0.3, 1.0, and 3.0 s. However, damage frequency did not correlate well with the spectral acceleration at periods of 1.0 and 3.0 s. Therefore, further analysis was performed only on the data for the spectral acceleration at a period of 0.3 s.

Analyses were conducted to identify: the effect of spectral acceleration; the combined effects of spectral acceleration and year of construction; and the combined effects of spectral acceleration and bridge type.

The percentage of the bridges that were damaged correlated well with the magnitude of the spectral acceleration at 0.3 s, as shown in Fig. 3-4. An exception to this trend was the decrease at the highest range of the spectral acceleration. This anomaly is most likely attributable to the small number of bridges in each category.
3.4.1 Combined effect of spectral acceleration and year constructed

Taking into account the year of construction further refined the spectral-acceleration analysis. As discussed in Section 3.1, bridges were classified into three categories according to the year of construction: before 1940, 1940–1975, and after 1975. The results of this analysis are displayed in Fig. 3-5.
As noted before, the bridges with high spectral accelerations were more likely to be damaged. Moreover, at each level of spectral acceleration, the bridges that were built before 1940 had the highest percentage of damaged bridges, and in general, those built after 1975 were the least likely to be damaged.
3.4.2 Combined effect of spectral acceleration and bridge type

The movable bridges were the most vulnerable type of bridge. Of the 42 movable bridges within the boundaries of the ShakeMap, six were damaged, resulting in an average damage percentage of 14%. Fig. 3-6 shows that the percentage of damaged movable bridges tended to increase with spectral acceleration. For example, of the nine bridges with estimated spectral accelerations above 0.4g, three (33%) were reported to have suffered damage. There was a notable exception to this trend. None of the eight movable bridges with estimated spectral accelerations in the range 0.30g to 0.40g were reported to suffer any damage. Such exceptions should be expected for small data sets.
Damage to the three types of immobile bridges (reinforced concrete, prestressed concrete and steel) were analyzed as a function of spectral acceleration. Settlement damage would not be expected to depend on bridge type. As a result, movable bridges (6), bridges with settlement (7), and bridges outside the limits of the ShakeMap (7) were not considered in this analysis. The results of the analysis for the remaining 58 bridges are reported in Fig. 3-7.
Despite the small number of damaged bridges in each category, Fig. 3-7 shows a clear correlation between the percentage of bridges that were damaged and the level of spectral acceleration. Based on this breakdown, it appears that the steel bridges were more vulnerable
than those constructed of reinforced or prestressed concrete. However, this observation is not attributable solely to the type of bridge, but to the year that the bridges were constructed, as shown in Fig. 3.8. As shown in this figure, both reinforced concrete and steel bridges that were constructed before 1940 were much vulnerable than bridges constructed later. In addition, 40% of the damage to steel bridges consisted of damage to the reinforced concrete substructure (Fig. 2-1).

Fig. 3-8: Combined effects of spectral acceleration, year of construction, and bridge type
4 Conclusions

The 2001 Nisqually earthquake damaged at least 78 bridges, of which 68 had slight or mild damage, and 10 had moderate damage. The most common type of reported damage (48 bridges) consisted of concrete cracking and spalling.

Reports of bridge damage were combined with the Washington State Bridge Inventory and ShakeMaps produced by the Pacific Northwest Seismic Network to identify factors that made bridges most vulnerable. If ShakeMaps were available immediately after an earthquake in the future, the results of this study could be used to prioritize post-earthquake inspections.

The percentage of bridges that were damaged did not correlate well with the distance from the bridge to the epicenter or the estimated peak ground acceleration at the bridge site. The estimated spectral acceleration at 0.3 s was a better indicator of the likelihood of bridge damage.

The year in which the bridge was constructed and the type of bridge were also important factors, with the highest percentages of damage reported for bridges that were built before 1940 and those that were movable. For estimated spectral accelerations above 0.4g, damage was reported to 33% of the movable bridges, 29% of the reinforced concrete bridges built before 1940, and 50% of steel bridges built before 1940. Although the damage percentage for bridges with a steel main span was generally higher than for other types of bridges, the number of such bridges was small, and the most common type of damage in these bridges was not to the steel superstructure, but rather, to the reinforced concrete substructure.
References


Appendix A: Distance Calculation

The distance from each bridge to the earthquake epicenter was calculated based on their respective latitudes and longitudes. By knowing the approximate radius of the earth, as well as the latitude and longitude of the point, it is possible to construct the spherical coordinates of this location on the earth’s surface. These can then be converted into Cartesian coordinates by the following set of equations.

\[ x = R \cos \phi \cos \theta \]
\[ y = R \cos \phi \sin \theta \]
\[ z = R \sin \phi \]

where \( R \) is the radius of the earth, \( \phi \) is the latitude and \( \theta \) is the longitude. \( \phi \) is positive above the equator, and \( \theta \) is considered positive if east of the International Date Line.

From the rectangular coordinates, the vector formed by connecting the origin to the point on the earth’s surface can be determined.

\[
P_1 = \begin{bmatrix} x_1 \\ y_1 \\ z_1 \end{bmatrix} \quad P_2 = \begin{bmatrix} x_2 \\ y_2 \\ z_2 \end{bmatrix}
\]

\( P_1 \) denotes the location of the epicenter, and \( P_2 \) denotes the location of the bridge.

The distance between these two points can be calculated as an arc along the earth’s surface, or as a straight line (chord) beneath the earth’s surface. Because the waves of an earthquake do not follow either of these exactly, and because both calculations would yield approximately the same answer, the arc-based measurement was chosen to estimate epicentral distances.

The angle between the two vectors can be determined by using the equation

\[ P_1 \cdot P_2 = \| P_1 \| \| P_2 \| \cos \beta \]

where \( \beta \) is the angle between the two vectors. Since both the points lay on the Earth’s surface, \( \| P_1 \| = \| P_2 \| = R \). Solving for \( \beta \),
\[ \beta = \cos^{-1}\left( \frac{P_1 \cdot P_2}{R^2} \right) \]  

(A-4)

Once the angle between the two vectors is known, the arc length between the two points can be determined by the equation

\[ D = R\beta \]  

(A-5)

where \( D \) is the distance between the epicenter and the point of interest. Combining Equations A-1, A-2, A-4 and A-5,

\[ D = R \cos^{-1} \left( \frac{\begin{bmatrix} R \cos \phi_1 \cos \theta_1 \\ R \cos \phi_1 \sin \theta_1 \\ R \sin \phi_1 \end{bmatrix} \cdot \begin{bmatrix} R \cos \phi_2 \cos \theta_2 \\ R \cos \phi_2 \sin \theta_2 \\ R \sin \phi_2 \end{bmatrix}}{R^2} \right) \]  

(A-6)

Simplifying the above equation, and using the trigonometric identity

\[ \cos(A - B) = \cos A \cos B + \sin A \sin B \]

the epicentral distance can be calculated as follows.

\[ D = R \cos^{-1} \left[ \left( \cos \phi_1 \cos \phi_2 \right) \left( \cos(\theta_1 - \theta_2) \right) + \sin \phi_1 \sin \phi_2 \right] \]  

(A-7)

With this equation, the distance from the epicenter to the point of interest can be directly linked to the latitude and longitude of the two points.
# Appendix B: Data Collection Inquiry Form

| Contact | |  |
|---------|---|  |
| Name:   | |  |
| Agency:            |  |
| Phone Number:      |  |
| Fax Number:        |  |

**AGENCY INFORMATION**

| Total Number of Bridges in Agency |  |
|----------------------------------|  |
| Total Number of Damaged Bridges in Agency |  |

**BRIDGE IDENTIFICATION**

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Bridge Number/Designation</th>
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</thead>
<tbody>
<tr>
<td>Latitude</td>
<td>Longitude</td>
</tr>
<tr>
<td>National Bridge Inventory Number</td>
<td></td>
</tr>
<tr>
<td>Physical Description of Location</td>
<td></td>
</tr>
</tbody>
</table>

**BRIDGE DAMAGE**

| Description of Damage |  |
|-----------------------|  |
| Have Photograph of Bridge (Y/N) | Have Photograph of Damaged Section (Y/N) |

**CONSEQUENCES OF DAMAGE**

| Duration of Closure |  |
|---------------------|  |
| Average Daily Traffic |  |
| Repair Date (Actual or Anticipated) |  |
| Cost of Repair |  |
| Bridge Value |  |

Please return care of
Marc O. Eberhard or R. Tyler Ranf
University of Washington Department of Civil Engineering.
Fax: (206) 543-1543  Phone: (206) 543-4815
Appendix C: Characteristics of Damaged Bridges

KEY

PGA = Estimated peak ground acceleration
PSA03 = Estimated spectral acceleration at a period of 0.3 s
PSA10 = Estimated spectral acceleration at a period of 1.0 s
PSA30 = Estimated spectral acceleration at a period of 3.0 s

Damage level = amount of damage ($)
sustained by the bridge
1 = $30,000 and under
2 = $30,001 - $100,000
3 = more than $100,000

Damage category = the type of damage that
the bridge sustained
1 = Settlement damage
2 = Concrete damage
3 = Steel damage
4 = Damage to restraints, bearings, or joints
5 = Damage to movable bridges

Main Span Material = the material for the load bearing member of the main span is made
2 = Reinforced concrete
4 = Steel
6 = Prestressed concrete

Main Span Design = the type of bridge
1 = Slab
2 = Stringer/multi-beam or girder
3 = Girder and floor beam system
4 = tee beam
5 = box beam/box girder – multiple
6 = Box beam/box girder – single or spread
7 = Rigid frame
8 = Orthotropic
9 = Truss – deck
10 = Truss – through
11 = Arch – deck
12 = Arch – through
13 = Suspension
14 = Stayed girder
15 = Movable – lift
16 = Movable – bascule
17 = Movable – swing
<table>
<thead>
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<th>Bridge Designation</th>
<th>Agency</th>
<th>Agency Bridge #</th>
<th>Washington State Bridge Inventory</th>
<th>Lat</th>
<th>Lon</th>
<th>distance to epicenter (km)</th>
<th>PGA</th>
<th>PSA03</th>
<th>PSA10</th>
<th>PSA30</th>
<th>Year Con</th>
<th>Damage Description</th>
<th>ADT</th>
<th>Estimated Repair Cost</th>
<th>damage level</th>
<th>damage category</th>
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<th>Main Span Des</th>
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<td>121.6433</td>
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<td>11.80</td>
<td>25.95</td>
<td>12.75</td>
<td>1.27</td>
<td>1963</td>
<td>Open (4&quot; drop @ approach, filled w/ACP by region)</td>
<td>8935</td>
<td></td>
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<td>1</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>090'10EB &amp; 90/10E-S</td>
<td>WSDOT-NW</td>
<td>90/10 E-S</td>
<td>0000000OC</td>
<td>47.5933</td>
<td>122.3200</td>
<td>57.33</td>
<td>11.79</td>
<td>31.93</td>
<td>17.81</td>
<td>2.36</td>
<td>1991</td>
<td>Closed. Damaged Bearings at joint connecting both bridges.</td>
<td>4406</td>
<td></td>
<td>1</td>
<td>4</td>
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<td>004/208</td>
<td>WSDOT-SW</td>
<td>004/208</td>
<td>0001442A</td>
<td>46.1917</td>
<td>123.1350</td>
<td>111.07</td>
<td>0.00</td>
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<td>0.00</td>
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<td>1930</td>
<td>Spalls on columns</td>
<td>3994</td>
<td></td>
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<td>1930</td>
<td>Spalls on columns</td>
<td>3994</td>
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<td>5</td>
<td>101/217 HOH River</td>
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<td>101/217</td>
<td>0001457A</td>
<td>47.8083</td>
<td>124.2483</td>
<td>135.62</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1931</td>
<td>Open (Banged against abut.)</td>
<td>1770</td>
<td></td>
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<td>2</td>
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<td>9</td>
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<td>6</td>
<td>101/418 Skokomish R.</td>
<td>WSDOT-OLYMPIC</td>
<td>101/418</td>
<td>0001604A</td>
<td>47.3117</td>
<td>123.1750</td>
<td>38.54</td>
<td>10.93</td>
<td>28.18</td>
<td>12.39</td>
<td>2.79</td>
<td>1932</td>
<td>Fractured corbel at exp. Jt. Main pier ahead on stationing, right side of bridge.</td>
<td>4758</td>
<td></td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>10</td>
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<td>7</td>
<td>012/15 Aberdeen Viaduct</td>
<td>WSDOT-OLYMPIC</td>
<td>012/15</td>
<td>0001679A</td>
<td>46.9583</td>
<td>123.7917</td>
<td>83.75</td>
<td>6.37</td>
<td>21.98</td>
<td>5.53</td>
<td>0.90</td>
<td>1994</td>
<td>Bent #3 SW column spall</td>
<td>20146</td>
<td></td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>4</td>
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<tr>
<td>8</td>
<td>002/37</td>
<td>WSDOT-NW</td>
<td>2/37</td>
<td>0001706A</td>
<td>47.8333</td>
<td>121.6417</td>
<td>110.50</td>
<td>8.91</td>
<td>20.04</td>
<td>7.76</td>
<td>0.72</td>
<td>1933</td>
<td>Open... EQ restrainers elongated ~2&quot; at pin/hanger locations of span 2. (4 locations)</td>
<td>6726</td>
<td>$15,000</td>
<td>1</td>
<td>4</td>
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<tr>
<td>9</td>
<td>002/39</td>
<td>WSDOT-NW</td>
<td>2/39</td>
<td>0001706C</td>
<td>47.8167</td>
<td>121.5967</td>
<td>111.77</td>
<td>11.55</td>
<td>25.00</td>
<td>13.05</td>
<td>1.37</td>
<td>1933</td>
<td>EQ restrainers @ P3 have 2-3&quot; gap between plates.</td>
<td>6726</td>
<td></td>
<td>1</td>
<td>4</td>
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<td>10</td>
<td>002/45 RR OC</td>
<td>WSDOT-NW</td>
<td>2/45 002059A 47.8033-121.5167 115.42 9.35 18.97 9.06 1.14 1936 Open. One broken longitudinal restrainer sheared off horizontal restrainer- sprung 23&quot; horizontally.</td>
<td>5308</td>
<td>1 4 2 4</td>
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<td>11</td>
<td>005/345E Nisqually</td>
<td>WSDOT-OLYMPIC</td>
<td>5/345E 002069A 47.0717-122.7033 9.04 14.91 60.68 32.62 12.93 1937 Open. Bent and broken cross frames, minor spalling to catcher blocks. Chipped paint around bearing shoes and pins. (see photos)</td>
<td>45830</td>
<td>$50,000 2 3 4 10</td>
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<td>12</td>
<td>012/12N Wishkah R.</td>
<td>WSDOT-OLYMPIC</td>
<td>12/12N 0002311A 46.9767-123.8083 84.46 6.72 24.08 6.23 0.96 1925 Paint pops around entrance portal sway brace rivets</td>
<td>15125</td>
<td>1 5 4 16</td>
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<td>13</td>
<td>Hylebos Waterway Bridge</td>
<td>Tacoma F04</td>
<td>0002376A 47.2778-122.3930 28.24 8.93 27.24 15.25 3.02 1939 Damage to leafs, temporary support required</td>
<td>$5,000 1 5 4 16</td>
<td></td>
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<td>14</td>
<td>016/110 Tacoma Narrows</td>
<td>WSDOT-OLYMPIC</td>
<td>16/110 0003418A 47.2617-122.5400 18.15 8.10 28.68 10.65 2.94 1943 Cable impact to anchor gallery penetration and minor spalling at impact area.</td>
<td>75337</td>
<td>$50,000 2 3 4 13</td>
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<td>303/4A Manette</td>
<td>WSDOT-OLYMPIC</td>
<td>303/4A 0003531A 47.5700-122.6217 46.85 18.28 44.47 11.04 1.24 1935 Damage to gusset span 7, pier 7</td>
<td>7300</td>
<td>$50,000 2 3 4 10</td>
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<td>16</td>
<td>433/1 Lewis and Clark</td>
<td>WSDOT-SW</td>
<td>433/1 0003760A 46.0983-122.9683 118.35 0.00 0.00 0.00 0.00 1929 Open. Temporary wooden blocks rotated and cracked</td>
<td>17871</td>
<td>1 3 4 10</td>
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<td>17</td>
<td>099/540 Alaska Way Viaduct</td>
<td>WSDOT-NW</td>
<td>99/540NB 0003935A 47.6000-122.3383 57.26 13.32 39.58 23.03 2.56 1952 099_540_99_540 EQ damage and repair.doc</td>
<td>35997</td>
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<td>005/308W</td>
<td>WSDOT-OLYMPIC</td>
<td>5/308W 0004544B 46.8967-122.9600 33.67 12.26 26.02 9.52 3.35 1954 Spalling of pier</td>
<td>25289</td>
<td>1 2 2 4</td>
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<td>005/321 Capital lake</td>
<td>WSDOT-OLYMPIC</td>
<td>5/321 0005090A 47.0233-122.9017 19.84 14.78 40.47 18.65 6.52 1956 Broken E.Q. Restrainers. Repair transverse stops at north abutment.</td>
<td>45733</td>
<td>$100,000 2 4 2 6</td>
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<td>Capitol Blvd</td>
<td>WSDOT-OLYMPIC</td>
<td>5/322 0005152A 47.0250-122.8983 19.53 14.78 40.47 18.65 6.52 1956 Bracing broken, bent bearing stiffeners. Damaged end Cross-frames &amp; bottom lateral bracing @ Tumwater end</td>
<td>14771</td>
<td>$100,000 2 3 4 11</td>
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<tr>
<td>109/6 Grass Creek</td>
<td>WSDOT-OLYMPIC</td>
<td>Pier 5 pitched on expansion joint, spilled over exposed rock</td>
<td>Pier 5 pitched at expansion joint, spilled over exposed rock</td>
<td>1966</td>
<td>$150,000</td>
<td>3</td>
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<td>005/197A</td>
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<td>Pier 24 at expansion joint</td>
<td>Pier 24 at expansion joint</td>
<td>1968</td>
<td>$150,000</td>
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<tr>
<td>509/5A Murray</td>
<td>WSDOT-OLYMPIC</td>
<td>Roadway open, but not to marine traffic</td>
<td>Roadway open, but not to marine traffic</td>
<td>1968</td>
<td>$150,000</td>
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<td>Open, pier 25 at expansion joint</td>
<td>Open, pier 25 at expansion joint</td>
<td>1968</td>
<td>$150,000</td>
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<tr>
<td>101/130 WSDOT-OLYMPIC</td>
<td>Pier 5 lived at expansion joint, spilled over exposed rock</td>
<td>Pier 5 pitched at expansion joint, spilled over exposed rock</td>
<td>Pier 5 pitched at expansion joint, spilled over exposed rock</td>
<td>1966</td>
<td>$150,000</td>
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<td>005/197A WSDOT-OLYMPIC</td>
<td>Pier 24 at expansion joint</td>
<td>Pier 24 at expansion joint</td>
<td>Pier 24 at expansion joint</td>
<td>1968</td>
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<td>512/1 I-5 OC WSDOT-OLYMPIC</td>
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<td>Roadway open, but not to marine traffic</td>
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<td>1968</td>
<td>$150,000</td>
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<td>099/538 Spokes St Viaduct WSDOT-OLYMPIC</td>
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Open. Per 5 piles 4.5, 6 have full perimeter cracks 4' above groundline. Per 4 pile 2 similar, with minor 0.5

5.9 405/50S
Satsop River
WSDOT-OLYMPIC 12/50S 0007612E 46.9983 -123.4967 61.08 12.54 35.27 16.66 4.39 1965

Open. Pier 5 piles 4, 5, 6 have full perimeter cracks 4' above groundline. Pier 4 pile 2 similar, with minor 0.5

Open. Cracks in center span closure pour and 2.5' deep. Soil liquefaction around columns of piles 2, 3, & 4. Sink holes up to 8' deep 2.5' 1966. Pier 10, 1 foot from NE corner 2.5' 1966. 500

Open. Bent and broken cross frames, minor spalling to catcher blocks. 26276 $20,000 2 3 4 10

Soil liquefaction around columns of piles 2, 3, & 4. Sink holes up to 8' deep 2.5' 1966. Pier 10, 1 foot from NE corner 2.5' 1966. 500

East ramp, north restrainer had broken grout pad. 8.1' vertical crack, 1/8" wide on girder G, outside face, 2.5' 1966. 2,522 1 2 6 2

Next inspection- crack in first diagonal from NE corner, 3" long 7 feet 1967.

Next inspection- crack in first diagonal from NE corner, 3" long 7 feet 1967.

Open. Bent and broken cross frames, minor spalling to catcher blocks. 45830 $50,000 2 3 4 10

Open. Pier 5 piles 4.5, 6 have full perimeter cracks 4' above groundline. Per 4 pile 2 similar, with minor 0.5

Open. Cracks in center span closure pour and 2.5' deep. Soil liquefaction around columns of piles 2, 3, & 4. Sink holes up to 8' deep 2.5' 1966. Pier 10, 1 foot from NE corner 2.5' 1966. 500

Open. Bent and broken cross frames, minor spalling to catcher blocks. 26276 $20,000 2 3 4 10

East ramp, north restrainer had broken grout pad. 8.1' vertical crack, 1/8" wide on girder G, outside face, 2.5' 1966. 2,522 1 2 6 2

Next inspection- crack in first diagonal from NE corner, 3" long 7 feet 1967.

Next inspection- crack in first diagonal from NE corner, 3" long 7 feet 1967.
<table>
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<th></th>
<th>Location</th>
<th>County</th>
<th>State Place</th>
<th>X (Westing)</th>
<th>Y (Northing)</th>
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<td>766</td>
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<td>Minor spalling at caps due to girder movement</td>
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<td>17.29</td>
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<td>1967</td>
<td>Crack in conc. Pile cap (1st cap south of north abutment, westside)</td>
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<td>57</td>
<td>4th Ave S., Jackson to Airport</td>
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<td>47.5993</td>
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<td>57.59</td>
<td>11.79</td>
<td>31.93</td>
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<td>1910</td>
<td>Spalls between building and sidewalk, also near Bent 30. Rails pulled apart.</td>
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<td>2ND AVE SOUTH EXTENSION</td>
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<td>7</td>
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<td>57.51</td>
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<td>2.36</td>
<td>1928</td>
<td>Spalls and cracks in the north west abutment</td>
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<td>1982</td>
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<td>60</td>
<td>E. Marginal Way at S. Horton St.</td>
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<td>100</td>
<td>08508600</td>
<td>47.5757</td>
<td>122.3381</td>
<td>54.96</td>
<td>20.06</td>
<td>60.22</td>
<td>47.43</td>
<td>4.38</td>
<td>1931</td>
<td>Settlement and cracking, sand boils and lateral spreading to the west.</td>
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<td>61</td>
<td>E. 26th St. Bridge</td>
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<td>20.53</td>
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<td>10.33</td>
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<td>2.23</td>
<td>1917</td>
<td>Cracks and spalling at expansion joints, outside edge beams.</td>
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<td>Emerson St. Viaduct</td>
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<td>17.86</td>
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<td>Seismic joint between 15th Ave W Interchange and Emerson St. Viaduct damaged.</td>
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<td>Vertical and shear cracks in column carrying expansion joint</td>
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<td>2</td>
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<td>68</td>
<td>08529800</td>
<td>47.5425</td>
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<td>1975</td>
<td>Damaged transverse shear block, spalling of cap beams</td>
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<td>2</td>
<td>6</td>
<td>4</td>
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<td>47.5717</td>
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<td>Docking Bearing/Shims failed, railing and expansion joint damage</td>
<td>$2,100,000</td>
<td>3</td>
<td>5</td>
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<td>Lakemont Boulevard Bridge</td>
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<td>BELLEVU11</td>
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Appendix D: List of Contributors

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Phone: (360) 753-8740

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PEER 2001/11  Analytical and Experimental Study of Fiber-Reinforced Elastomeric Isolators. James M. Kelly and Shakhzod M. Takhirov. September 2001. $15.00


PEER 2001/06  Development of an Electrical Substation Equipment Performance Database for Evaluation of Equipment Fragilities. Thalia Agnanos. April 1999. $15.00

PEER 2001/05  Stiffness Analysis of Fiber-Reinforced Elastomeric Isolators. Hsiang-Chuan Tsai and James M. Kelly. May 2001. $20.00

PEER 2001/04  Organizational and Societal Considerations for Performance-Based Earthquake Engineering. Peter J. May. April 2001. $15.00


PEER 2001/01  Experimental Study of Large Seismic Steel Beam-to-Column Connections. Egor P. Popov and Shakhzod M. Takhirov. November 2000. $15.00
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<td>PEER 1999/10</td>
<td>U.S.-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures.</td>
<td>December 1999.</td>
<td>$33.00</td>
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Adoption and Enforcement of Earthquake Risk-Reduction Measures. Peter J. May, Raymond J. Burby, T. Jens Feeley, and Robert Wood. $15.00

Task 3 Characterization of Site Response General Site Categories. Adrian Rodriguez-Marek, Jonathan D. Bray, and Norman Abrahamson. February 1999. $20.00


Behavior and Failure Analysis of a Multiple-Frame Highway Bridge in the 1994 Northridge Earthquake. Gregory L. Fenves and Michael Ellery. December 1998. $20.00


Rocking Response and Overturning of Equipment under Horizontal Pulse-Type Motions. Nicos Makris and Yiannis Roussos. October 1998. $15.00


Repair/Upgrade Procedures for Welded Beam to Column Connections. James C. Anderson and Xiaojing Duan. May 1998. $33.00
PEER 1998/02  *Seismic Evaluation of 196 kV Porcelain Transformer Bushings.* Amir S. Gilani, Juan W. Chavez, Gregory L. Fenves, and Andrew S. Whittaker. May 1998. $20.00

PEER 1998/01  *Seismic Performance of Well-Confined Concrete Bridge Columns.* Dawn E. Lehman and Jack P. Moehle. December 2000. $33.00