



**STRUCTURAL SYSTEMS
RESEARCH PROJECT**

Report No.
SSRP-2001/02

**RECONNAISSANCE REPORT OF THE
FEBRUARY 28, 2001 NISQUALLY
(SEATTLE-OLYMPIA) EARTHQUAKE**

by

**André Filiatrault
Chia-Ming Uang
Bryan Folz
Constantin Chrstopoulos
and
Kip Gatto**

Report of a field reconnaissance sponsored by the Pacific Earthquake Engineering Research (PEER) Center and the Consortium of Universities for Earthquake Engineering (CUREE)

March, 2001

Department of Structural Engineering
University of California, San Diego
La Jolla, California 92093-0085

Report No. SSRP-2001/02

**RECONNAISSANCE REPORT OF THE
FEBRUARY 28, 2001 NISQUALLY (SEATTLE-OLYMPIA)
EARTHQUAKE**

by

**André Filiatrault
Professor of Structural Engineering**

**Chia-Ming Uang
Professor of Structural Engineering**

**Bryan Folz
Visiting Associate Professor**

**Constantin Christopoulos
Graduate Student Researcher**

and

**Kip Gatto
Graduate Student Researcher**

Department of Structural Engineering
University of California, San Diego
La Jolla, California 92093-0085

March, 2001

DISCLAIMER

Opinions, findings, conclusions and recommendations expressed in this report are those of the authors. The Pacific Earthquake Engineering Research Center and the Consortium of Universities for Research in Earthquake Engineering assume no liability for the information included in this report.

ACKNOWLEDGEMENTS

The Pacific Earthquake Engineering Research (PEER) Center and the Consortium of Universities for Research in Earthquake Engineering (CUREE) sponsored this preliminary field reconnaissance report. Dr. Jyr-Jong Lee of CES Inc. and Dr. Hongzhi Zhang of Washington State Department of Transportation provided background information on the bridge structures described in this report.

We greatly appreciated the input and coordination provided by Professor Greg Deierlein, Deputy Director of PEER, from Stanford University and by Robert Reitherman, Executive Director of CUREE.

TABLE OF CONTENTS

DISCLAIMER	i
ACKNOWLEDGEMENTS	ii
TABLE OF CONTENTS	iii
1. INTRODUCTION	1
2. STRONG GROUND MOTIONS	3
3. PERFORMANCE OF BUILDING STRUCTURES	8
3.1 Performance of Masonry and Wood Buildings	8
3.2 Performance of Steel Buildings	28
3.3 Performance of Reinforced Concrete Buildings	33
4. PERFORMANCE OF BRIDGE STRUCTURES.....	37
4.1 Holgate Bridge	37
4.2 Spokane St. Overcross (SR 99).....	40
4.3 Spokane St. Viaduct at 4 th Avenue	44
4.4 Fourth Avenue Bridge in Olympia	46
5. PERFORMANCE OF NON-STRUCTURAL COMPONENTS.....	50
5.1 Performance of Ceiling Systems.....	50
5.2 Performance of Interior Wall Finishes.....	52
5.3 Performance of Exterior Wall Finishes.....	55
5.4 Performance of Window Systems.....	56
5.5 Performance of Building Contents.....	57
5.6 Performance of Gas and Water Systems.....	60
5.7 Performance of Electrical Systems	61
6. CONCLUSIONS	62

1. INTRODUCTION

On February 28, 2001 an earthquake of moment magnitude $M_w = 6.8$ struck the Puget Sound in the western region of Washington State. The main shock of this seismic event occurred at 10:54 am (PST) and originated at a depth of 52 km. As illustrated in Fig. 1.1, the epicenter of the earthquake was located in the Nisqually Valley at a distance of about 20 km northeast of Olympia and 60 km southwest of Seattle.



Figure 1.1 Epicentral Location of Nisqually Valley Earthquake (courtesy of USGS).

The Washington Emergency Management Division tallied more than 400 injuries directly related to the earthquake. No fatalities were directly attributable to the earthquake. Preliminary reports obtained 24 hours after the earthquake estimated that damage had reached \$ 2 billion.

Initial seismological assessment associated this earthquake to the subduction seismotectonic mechanism between the Juan de Fuca and the North American plates. Similar strong subduction earthquakes occurred in the region in 1949 ($M_w = 7.1$) and in 1965 ($M_w = 6.5$). The Nisqually earthquake was the seventh in the region that recorded a magnitude above 6.0 since

1872. Consistent with deep subduction plate mechanisms, only two aftershocks of magnitude higher than 2.5 were recorded during the first 48 hours of the main shock.

As of March 3, 2001, 33 buildings had been red-tagged and 45 had been yellow-tagged in Seattle and Olympia as a result of the earthquake. Also, more than 25 bridges were closed for inspection or repair. Most of the reported damage was concentrated in the Seattle and Olympia areas with little damage reported in Tacoma, which is located about 30 km south of Seattle. About 200,000 customers lost electrical power when the ground shaking tripped circuit breakers. Most of the outages were in south King, Pierce and Thurston Counties. Limited damage was reported to other lifeline facilities. Only one fire erupted at the Cedar Creek Correction Center as a result of the earthquake.

Following the Nisqually earthquake, the authors were mandated by the Pacific Earthquake Engineering Research (PEER) Center and by the Consortium of Universities for Research in Earthquake Engineering (CUREE) to travel to the earthquake area and to provide a preliminary assessment of the performance of buildings, bridges and non-structural components. The field reconnaissance took place on March 2 to 4, 2001. The main objective of this preliminary reconnaissance report is to present and discuss these observations.

2. STRONG GROUND MOTIONS

Strong ground motion records for the Nisqually earthquake were available from both the United States Geological Survey (USGS) and the Pacific Northwest Seismograph Network (PNSN), centered at the University of Washington. Figure 2.1 shows the location of strong ground motion instruments in the western region of Washington State.

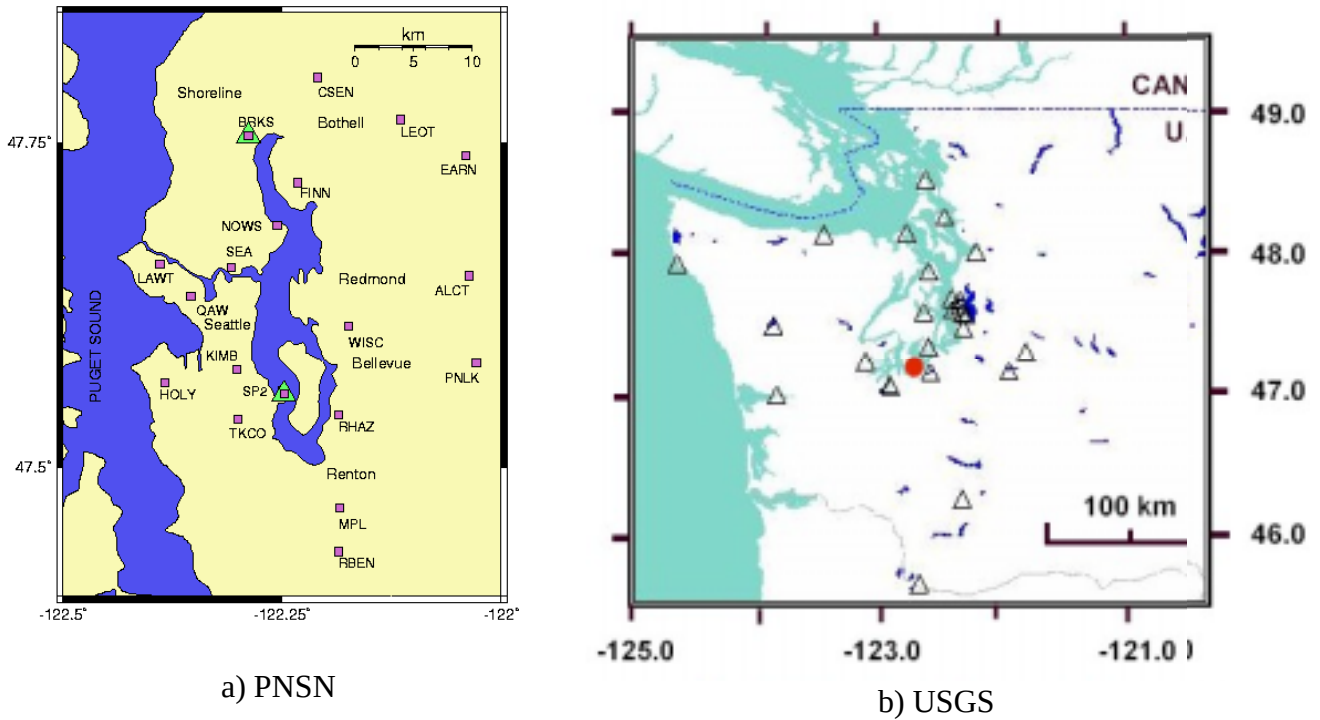


Figure 2.1 Locations of Strong Motion Seismographs in Western Washington State.

To provide information for the level of ground shaking in the areas that the team visited in this reconnaissance, the corrected ground acceleration time-histories and the associated pseudo-acceleration response spectra with 5% damping at five locations (see Fig. 2.2) are presented below. For the purpose of comparison, the elastic design response spectrum, based on the 2000 International Building Code¹ for soil type D, is also presented. The amplitude of this design spectrum is 2/3 of the amplitude of the Maximum Credible Earthquake (MCE) for the region (corresponding to a return period of 2500 years).

¹ "International Building Code. (2000)" International Code Council, Building Officials and Code Administrators International Inc., Country Club Hills, IL., International Conference of Building Officials, Whittier, CA., Southern Building Code Congress International, Inc., Birmingham, AL.

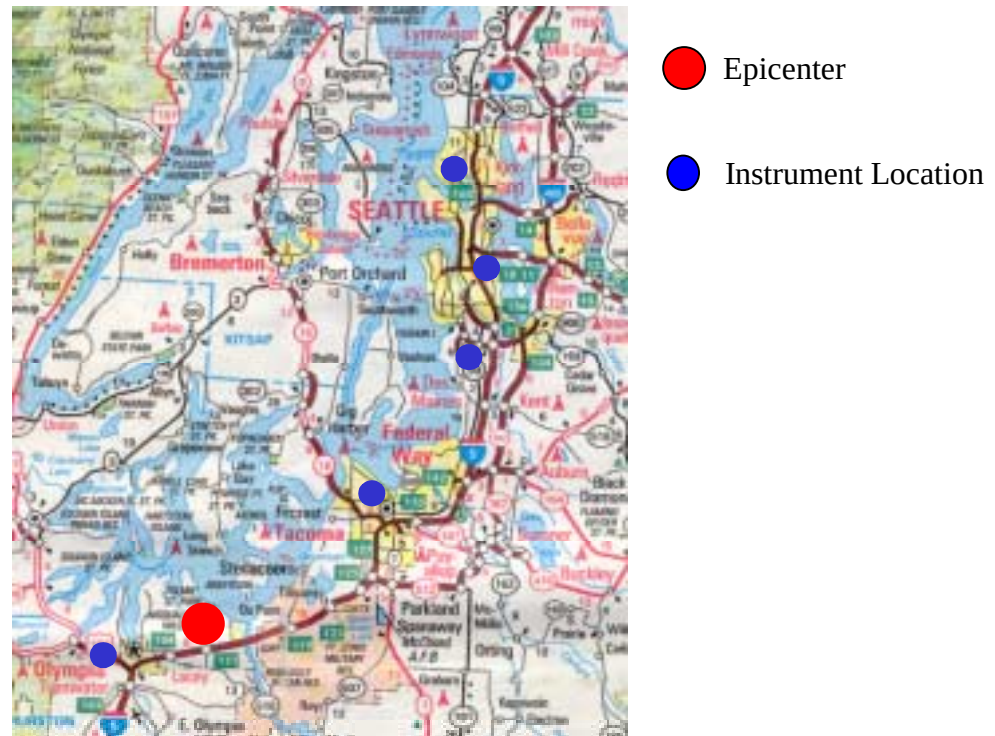


Figure 2.2 Location of Five Strong Motion Seismographs.

Olympia: WSDOT Test Laboratory

The seismograph, instrumented by USGS at the Test Laboratory of the Washington State Department of Transportation (WSDOT), is located 18 km from the epicenter. Figure 2.3 shows the ground acceleration time histories in the N-S and E-W directions. The peak ground acceleration (PGA), which occurred in the N-S direction, reached 0.25g. The duration of strong motion is relatively long, being of the order of 30 seconds. The response spectra in Fig. 2.3c indicate that the dominant frequency content of the ground motions is between 0.1 and 0.8 seconds. It is of interest to note from Fig. 2.3c that the N-S component of the earthquake has a spectral acceleration that matches the design spectrum at a period of approximately 0.7 second. Consequently, structures with a fundamental period in the range of 0.7 second would be subjected to the design level of ground shaking.

Tacoma: University of Puget Sound

This station, maintained by PNSN, is located 22 km away from the epicenter. Despite the long duration of shaking, Fig. 2.4 shows that the intensity is low, with a PGA value of 0.06g. The

response spectra show that the intensity of ground shaking is about one-fifth the design earthquake in the period range up to 0.8 second. The low level of ground shaking correlates with the minimal damage that was observed in Tacoma.

SeaTac International Airport

The air traffic control tower of the Seattle-Tacoma International Airport sustained significant damage during the earthquake. A USGS seismograph at the airport is approximately 45 km from the epicenter. The time histories in Fig. 2.5 show that the PGA reached 0.19g. A comparison of the response spectra shows that the intensity of ground shaking is about half that of the design earthquake.

Seattle: Kimball Elementary School

PNPSN installed a seismograph at Kimball Elementary School. Two bridges that suffered damage (see Sections 4.1 and 4.2) during the earthquake are located within 3 km from this station. With an epicentral distance of 57 km, Fig. 2.6 shows that the component in the E-W direction is stronger, and the PGA reached 0.14g. The response spectrum plots indicate that the intensity of the ground shaking was about one-third to one-quarter that of the design earthquake.

Seattle: Crowne Plaza Hotel

The Crowne Plaza Hotel is located in downtown Seattle with an epicentral distance of 58 km. USGS instrumented this 33-story building at four levels. Although the records retrieved from the basement are not free field ground motions, they do provide an indication of the level of ground shaking in downtown Seattle. Ground shaking near the waterfront is expected to be higher due to the soil effect. The time history plots in Fig. 2.7 show low-intensity ground shaking with a PGA value of 0.09g. The response spectrum plots indicate that the intensity of the ground shaking is about one quarter that of the design earthquake.

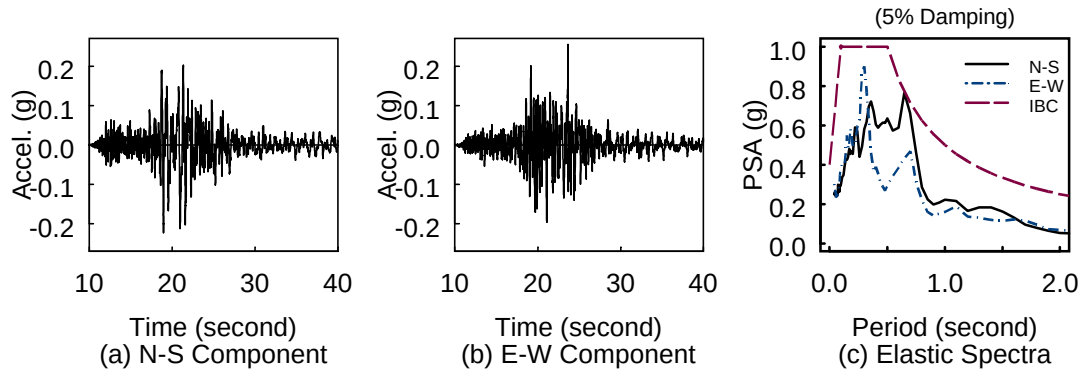


Figure 2.3 Olympia: WSDOT Test Laboratory.

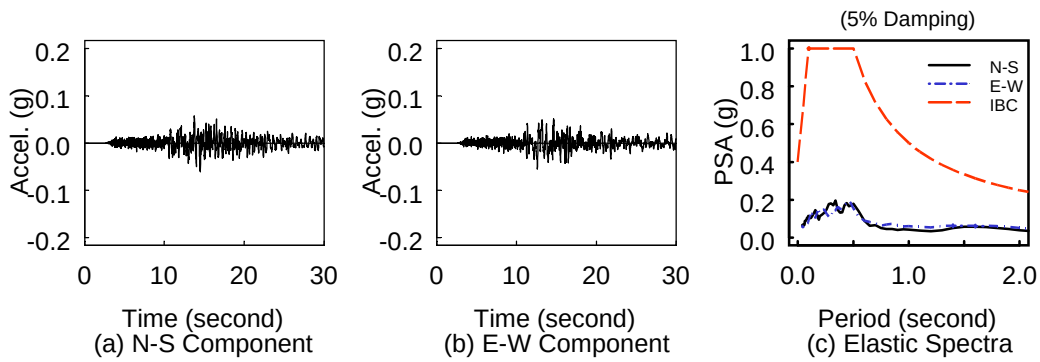


Figure 2.4 Tacoma: University of Puget Sound.

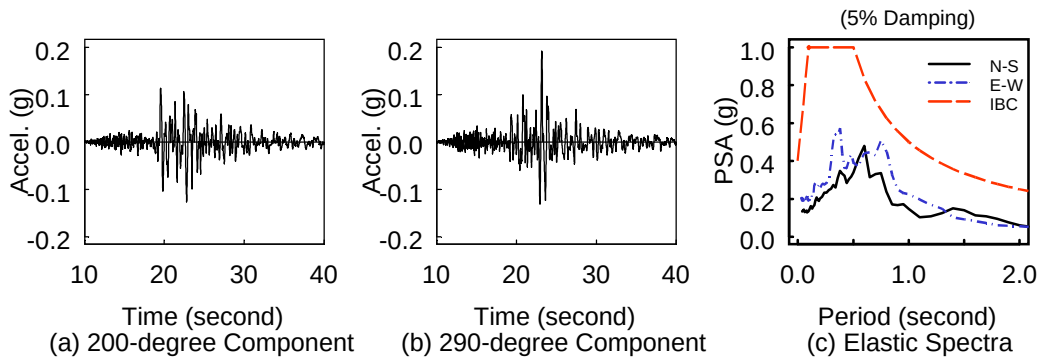


Figure 2.5 Seattle-Tacoma International Airport.

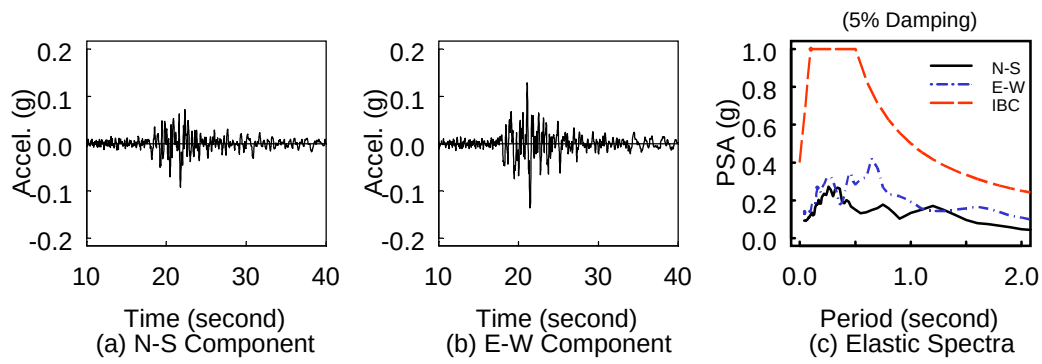


Figure 2.6 Seattle: Kimball Elementary School.

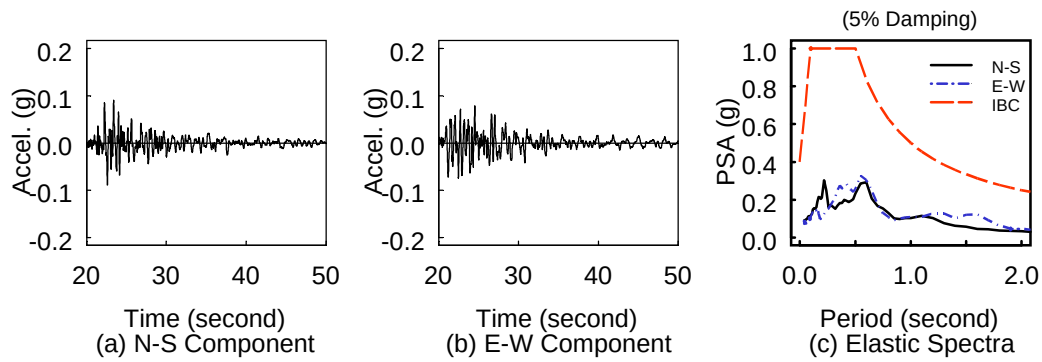


Figure 2.7 Seattle: Crowne Plaza Hotel.

3. PERFORMANCE OF BUILDING STRUCTURES

Reports in the first 72 hours of the Nisqually earthquake indicated that more than 80 buildings had suffered minor to moderate structural damage. About a dozen buildings sustained substantial structural damage requiring closure for a significant period of time. No building was scheduled for demolition at the time of writing. This section discusses the preliminary observations made on the performance of building structures during the field reconnaissance.

3.1 Performance of Masonry and Wood Buildings

Older low-rise unreinforced masonry buildings suffered the most structural damage as the result of the Nisqually earthquake. Most of the failures involved in-plane diagonal cracking and out-of-plane failure of masonry walls and parapets. Chimney failures were also observed. In some cases, these failures resulted in partial collapse of roof structures. Damage to wood buildings was light and was mainly associated with cracking of exterior wall finish materials and masonry foundations.

The most publicized masonry structure that suffered damage during the Nisqually earthquake was the legislative building in Olympia, which houses offices of the Governor and other elected officials. This building, shown in Fig. 3.1.1, was completed in 1927. The central part of the building consists from bottom to top of a lower concrete-masonry structure, a cylindrical wall system (called the drum), a cylindrical buttress system and a ribbed masonry dome.



Figure 3.1.1 Legislative Building in Olympia.

Damage to this structure is clearly evident from the severe cracking of the nonstructural sandstone pilaster from one of the exterior masonry buttresses supporting the dome, as shown in Fig. 3.1.2. This dislodged pilaster seemed wedged against the top portion of the structure, which prevented it from falling. Also at the time of writing, cracking had just been discovered in some of the ribs of the dome.



Figure 3.1.2 Severe Cracking of a Nonstructural Sandstone Pilaster at the Dome of the Legislative Building in Olympia.

The building was seismically upgraded in 1972 by reinforcing the masonry walls of the drum by a 600 mm thick cylindrical reinforced concrete wall. The lower portion of the building was

first seismically upgraded in 1965 and again in 1972. This lower portion is not structurally connected to the dome.

Significant differential movement occurred between the drum and the lower masonry portion of the building, as shown in Fig. 3.1.3. Preliminary engineering reports indicated that the drum wall would have deformed as an oval and also rocked during the ground shaking. Furthermore, the dome itself would have deflected on one side by about 75 mm and also rotated. Note that similar movements of the dome were also reported following the last two major earthquakes that struck the region in 1949 and in 1965.



Figure 3.1.3 Differential Movement between the Drum and Lower Masonry Portion of the Legislative Building in Olympia.

As a result of the movement of the drum and the dome, horizontal cracking of the architectural plaster occurred at the base of each interior masonry buttress, as shown in Fig. 3.1.4. At the time of writing, preparations were underway to investigate the structural condition of these interior buttresses that were believed to contain encased structural steel sections.



Figure 3.1.4 Cracking of Architectural Plaster at the Base of an Interior Masonry Buttress of the Legislative Building in Olympia.

Also field investigations by others were in progress to estimate the movement that took place between the dome and the buttresses during the earthquake and to determine the appropriate remedial action. Note that the next seismic upgrade of the building that was planned before the Nisqually earthquake occurred centered on connecting the dome to the main structure.

The Ramada Inn at 621 Capitol Way in Olympia was red-tagged because of differential vertical movement that was observed in the concrete floor slab at each story level of the building following the earthquake. Figure 3.1.5 shows a general view of the eight-story reinforced masonry structure, which is built on top of a two level reinforced concrete parking structure. Construction of the Ramada Inn was reported to have taken place circa 1970. As evident in Fig. 3.1.5, the structural system is comprised of a parallel grid of reinforced masonry walls that divide the guest rooms along each level. Two reinforced masonry stairwells and two elevator shafts in conjunction with corridor walls provide the lateral load-resisting system for the building.



Figure 3.1.5 General View of the Ramada Inn at 621 Capitol Way in Olympia.

Figure 3.1.6 shows an elevation view of the west end of the Ramada Inn. This section of the building settled relative to the remainder of the structure. At the eighth floor level, this differential settlement was of the order of 20 to 40 mm. It is of interest to note that the two stairwells and elevator shafts were east of this section of the building. It was observed that the settlement initiated along a construction joint that was located in the floor slabs at each level of the building. Vertical differential movement at the construction joint in the eighth floor level slab was of the order of 8 to 10 mm, as shown in Fig. 3.1.7.



Figure 3.1.6 Section of the Ramada Inn in Olympia Showing Evidence of Differential Vertical Movement at Each Floor Level.



Figure 3.1.7 Differential Vertical Movement in the Floor Slab on the Eight Floor of the Ramada Inn.

Minor spalling and cracking was also observed in the reinforced concrete columns in the parking structure under the Ramada Inn, as shown in Fig. 3.1.8.



Figure 3.1.8 Spalling of a Reinforced Concrete Column in the Parking Structure Under the Ramada Inn.

The Ramada Inn also sustained water damage from a ruptured supply line to a large water tank in the mechanical room on the roof of the building. This damage is discussed further in Section

5.6. As well, there was significant damage to the building contents as a result of the earthquake, as discussed further in Section 5.5.

Figure 3.1.9 shows the out-of-plane failure of an old single-story masonry warehouse incorporating timber roof trusses on Second Avenue near Harrison Street in the city of Kent. The timber roof trusses were also damaged but did not collapse. A shear failure in one of the perimeter lintels was also observed, as shown in Fig. 3.1.10.



Figure 3.1.9 Out-of-Plane Failure of an Old Single-Story Masonry Warehouse in Kent.



Figure 3.1.10 Shear Failure of a Perimeter Lintel of an Old Single-Story Masonry Warehouse in Kent.

A substantial out-of-plane failure of an unreinforced masonry exterior wall occurred in a two-story masonry and wood structure on Utah Street in downtown Seattle, as shown in Fig. 3.1.11.

The interior timber trusses and columns performed well during the earthquake and continued to support the roof structure even after the failure of the exterior wall.



Figure 3.1.11 Failure of Masonry Facade of a Two-Story Wood Building in Seattle.

Another similar out-of-plane failure occurred at the Seattle Chocolate Factory in downtown Seattle. The west parapet of this one-story masonry building collapsed onto the street, as shown in Fig. 3.1.12. Also, the south masonry façade of the building collapsed onto and failed the wood roof covering the loading dock, as shown in Fig. 3.1.13.



Figure 3.1.12 Collapse of the West Parapet of the Seattle Chocolate Factory in Seattle



Figure 3.1.13 Partial Collapse of the South Façade and Failure of the Loading Dock Canopy of the Seattle Chocolate Factory in Seattle.

Figure 3.1.14 shows the complete collapse of an unreinforced brick wall along the whole north exterior face of the Acme Tool and Specialty Company Building, situated on First Avenue South in Seattle. The timber beams and columns remaining intact during the earthquake prevented collapse of the timber roof structure.



Figure 3.1.14 Complete Failure of an Unreinforced Brick Wall along the North Face of the Acme Tool and Specialty Building on First Avenue South in Seattle.

Figure 3.1.15 shows an out-of-plane failure of a large section of an unreinforced brick wall in a red-tagged three-story masonry building in the 500 block of First Avenue South in Seattle.



Figure 3.1.15 Out-of-Plane Failure of a Section of a Masonry Wall in a Three-Story Building in the 500 Block of First Avenue South in Seattle.

Figure 3.1.16 shows another example of an out-of-plane failure of an unreinforced exterior masonry wall in the Skookum Bay Outfitters Building at 518 Capitol Way South in Olympia. Again, the interior wood framing performed well during the earthquake and continued to support the roof even with partial collapse of the exterior wall.



Figure 3.1.16 Partial Collapse of an Exterior Unreinforced Brick Wall in the Skookum Bay Outfitter Building at 518 Capitol Way South in Olympia.

Figure 3.1.17 shows the separation of perpendicular unreinforced masonry walls in a building in the 400 block of Second Avenue in Seattle. The permanent gap that opened between the walls resulted from falling bricks lodging in the crack during the earthquake.



Figure 3.1.17 Permanent Gap between Perpendicular Walls in a Masonry Building along the 400 Block of Second Avenue of Seattle.

Damage from the pounding of adjacent closely spaced building with different roof levels was commonly observed in the Pioneer Square area of Seattle. Figure 3.1.18 shows the partial failure of the roof structure and brick façade of a building in the 500 block of First Avenue South in Seattle, which mostly likely resulted from pounding against the adjacent building. Figure 3.1.19 shows that the brick façade at the end of this building was forced out of plane along the full building height. Masonry anchors through the brick façade helped to prevent a larger out-of-plane deformation of the wall.



Figure 3.1.18 Partial Roof Failure Resulting from Pounding of Adjacent Building in the 500 Block of First Avenue South in Seattle.



Figure 3.1.19 Brick Facade Forced Out-of-Plane by Pounding against an Adjacent Building in the 500 Block of First Avenue South in Seattle.

Another example of localized pounding damage, in the form of a shear crack, to a brick façade of a large four-story apartment building at 515 Occidental Avenue in Seattle is shown in Fig 3.1.20. The remainder of this building did not show any visual evidence of damage.



Figure 3.1.20 Localized Pounding Damage to a Brick Facade of a Four-Story Apartment Building at 515 Occidental Avenue in Seattle.

Significant in-plane cracking of masonry walls was also observed in several structures. One example of this occurred at the First Christian Church located on the corner of Seventh Avenue and Franklin Street in Olympia, as shown in Fig. 3.1.21. Diagonal cracking occurred between the large window openings and the re-entrant corners of the structure.



Figure 3.1.21 In-plane Cracking of Masonry Walls of the First Christian Church in Olympia.

Another example of in-plane cracking of masonry walls occurred in the east wing of Lincoln Elementary School in Olympia, as shown in Fig 3.1.22. This damage caused the building to be yellow-tagged and forced relocation of the students to a nearby church facility until repairs are

completed. As shown in Fig 3.1.22, repair to the structure was already underway at the time of this field reconnaissance.



Figure 3.1.22 In-plane Cracking of Masonry Walls in the East Wing of Lincoln Elementary School in Olympia.

Failures of parapets were observed in numerous masonry buildings throughout Olympia and Seattle. In tallying damage to unreinforced masonry and brick-faced buildings, total or partial collapse of parapets was the most common type of visual damage observed during the field reconnaissance. One example of this is the failure of a step parapet in a two-story military dormitory in Fort Lewis, as shown in Fig. 3.1.23. Parts of the parapet fell onto the roof of the building and punctured it. The building also sustained water damage as a result of heavy rain that occurred the day after the earthquake.



Figure 3.1.23 Failure of a Stepped Masonry Parapet in a Two-Story Military Dormitory Building in Fort Lewis.

Another example of an unreinforced parapet failure is shown in Fig 3.1.24 for the building at 558 First Avenue South in Seattle. With this building three quarters of the parapet along the west face of the building collapsed to the street below during the earthquake.



Figure 3.1.24 Collapse of an Unreinforced Parapet to the Building at 558 First Avenue South in Seattle.

A number of unbraced chimney failures were observed in older residential wood frame buildings in Olympia. An example of chimney failure is shown in Fig. 3.1.25 for a three-story apartment building in Olympia.



Figure 3.1.25 Chimney Failure in a Three-Story Wood Frame Apartment Building in Olympia.

Figure 3.1.26 shows the failure of another chimney in a three-story wood frame house located at the corner of Tenth Avenue and Franklin Street in Olympia. The top part of the chimney above the roof collapsed to the ground, while the bottom part separated from the house over its entire height.



Figure 3.1.26 Separation and Failure of the Chimney of a Three-Story Wood Frame House in Olympia.

Figure 3.1.27 shows a similar chimney failure that was also observed in a two-story wood frame house located at the corner of Capital Way and Twenty-Fourth Avenue in Olympia.



Figure 3.1.27 Chimney Failure in a Two-Story Wood Frame House in Olympia.

Braced chimneys, on the other hand, performed very well as shown in Fig. 3.1.28 for one of the chimneys of a wood frame army barrack in Fort Lewis.



Figure 3.1.28 Undamaged Braced Chimney of a Wood Frame Army Barrack in Fort Lewis.

Another type of damage widely observed in wood frame houses in Olympia was the cracking of concrete or masonry foundation walls. The cracking often originated in the corners of basement window openings. An example of this is shown in Fig. 3.1.29 for a house near Franklin Street in Olympia.



Figure 3.1.29 Cracking of the Concrete Foundation Wall of a Wood Frame House in Olympia.

An example of a seismically upgraded wood and masonry structure that performed well during the Nisqually earthquake is the four-story Cobalt Group Building in downtown Seattle, as shown in Fig. 3.1.30.



Figure 3.1.30 General View of the Cobalt Group Building in Downtown Seattle.

The original structural system of this building consisted of wood post-and-beam framing along with perimeter masonry walls. As shown in Fig. 3.1.31, the post-and beam connections have been upgraded with steel column caps and restraining straps.



Figure 3.1.31 Original Wood Post-and-Beam Framing System of the Cobalt Group Building in Downtown Seattle.

The seismic upgrade of the building also incorporated diagonal tubular steel cross-bracing systems, as shown in Fig. 3.1.32.



Figure 3.1.32 Diagonal Tubular Cross-Braces to Seismically Upgrade the Cobalt Group Building in Downtown Seattle.

Also in the basement of the building, reinforced concrete shear walls were incorporated as in-fill to the existing reinforced concrete vertical load bearing frames, as shown in Fig. 3.1.33.



Figure 3.1.33 Added Reinforced Concrete Shear Walls in the Basement of the Cobalt Group Building in Downtown Seattle.

Finally, tie-rods were introduced to connect the perimeter masonry walls to the floor and roof diaphragms of the building. As the result of this seismic upgrade, damage to the building was

limited to in-plane diagonal cracking of the east corner of the north masonry wall, as shown in Fig. 3.1.34.



Figure 3.1.34 In-plane Cracking of the North Masonry Wall of the Cobalt Group Building in Seattle.

During the earthquake, the building experienced water damage in its basement as a result of a water main breaking. Evidence of rocking of the base of the building in the east-west direction was also observed, as shown in Fig. 3.1.35.



Figure 3.1.35 Separation between West Facade of the Cobalt Group Building in Seattle and the Sidewalk Showing Evidence of Rocking of the Building in East-West Direction.

3.2 Performance of Steel Buildings

No report of structural damage to steel buildings was available at the time of the field reconnaissance. As a consequence, only three steel structures were visited during the field reconnaissance.

The first steel structure was the Kent Regional District Center as shown in Fig. 3.2.1. This five-story structure was completed in 1997 and incorporates a circular rotunda connected to two orthogonal rectangular wings.



Figure 3.2.1 Kent Regional District Center.

As shown in Fig 3.2.2, the lateral load resisting system of the structure consists of perimeter chevron eccentrically braced steel frames bearing on a reinforced concrete column system at the basement level. The bracing system behaved well with no sign of distress. Cracking of one of the reinforced concrete column in the basement was reported but was not considered detrimental to the normal operation of the building. The bracing system fully protected the large glass areas of the rotunda and all windows of the building.



Figure 3.2.2 Chevron Eccentric Bracing System at Kent Regional District Center.

Figure 3.2.3 shows evidence of pounding between the rotunda and the parapet of one of the adjacent wings. The vertical cracking of the parapet was also visible inside the building.



Figure 3.2.3 Evidence of Pounding at Kent Regional District Center.

The second building incorporating a steel lateral load-resisting system that was investigated is the Starbucks headquarter on Utah Street in downtown Seattle. The east facade of this nine-story building is shown in Fig. 3.2.4.



Figure 3.2.4 East Facade of the Starbucks Headquarter in Downtown Seattle.

The original structural system of this building consisted of a reinforced concrete flat slab-columns framing system with masonry walls. Parking and retail structures were later added next to the north and south sides of the building, respectively. Significant movement between the adjacent parts of the building occurred during the earthquake, as evidenced by the distortion of the vertical expansion joint between the original building and the north parking structure. The building was seismically upgraded circa 1995 by adding two lines of chevron eccentrically braced steel frames in both perpendicular directions. A general view of the bracing element on the sixth floor of the building is shown in Fig. 3.2.5.



Figure 3.2.5 Chevron Eccentrically Braced Steel Frame Used to Seismically Upgrade the Starbucks Headquarter Building in Downtown Seattle.

The eccentric braced frames in the north-south direction did not show any evidence of inelastic deformation while the central shear link of each bracing element in the east-west direction yielded, as shown in Fig. 3.2.6. This observation indicates that despite its fairly square floor plan, the dynamic response of the building was significantly larger in the east-west direction than in the north-south direction.



Figure 3.2.6 Yielding of a Chevron Eccentrically Braced Frame in the East-West Direction of the Starbucks Headquarter in Downtown Seattle.

The third steel building surveyed was the new air traffic control tower at Sea-Tac International Airport. While the existing control tower sustained significant damage as a result of the Nisqually earthquake and had to be closed, as shown in Fig. 3.2.7, the steel frame construction for the new 81 m tall control tower just north of the existing tower, which had been completed only a month before the earthquake, was observed to have sustained no damage, as shown in Fig. 3.2.8. The new tower's structural system is a mixed construction design. The bottom eleven stories are composed of 500 mm thick pre-cast concrete panel shear walls that are integrally connected to four cast-in-place reinforced concrete column boundary elements strengthened with an embedded wide-flange steel section at the corners. Composite concrete ring beams with embedded wide flange steel sections are used at the transition between the base structure and the steel frame system above.



Figure 3.2.7 Damage to the Existing Sea-Tac Air Traffic Control Tower.



Figure 3.2.8 Global View of New Sea-Tac Air Traffic Control Tower.

The new tower's base structure is transitioned into a 5-story steel frame structure at level 12 (47 m above the ground). The proprietary biaxial SidePlate dual system consisting of a Special Concentrically Braced Frame and a Special Moment Resisting Frame was adopted as the lateral load-resisting system. Each column is a built-up cruciform. A super X-braced configuration is used in the lower four stories, as shown in Fig. 3.2.9, while a Chevron-braced configuration is used in the top story.



Figure 3.2.9 Super X-Braces for Special Concentrically Braced Frame.

3.3 Performance of Reinforced Concrete Buildings

Structural damage observed in reinforced concrete buildings consisted mostly of diagonal shear cracking of lateral load-resisting shear walls. An example of this type of damage was observed in the five-story General Administration Building in Olympia, as shown in Fig. 3.3.1.



Figure 3.3.1 General Administration Building in Olympia.

The lateral load resisting system in the north-south direction consists of pairs of shear walls along the east and west faces of the building. On each face, one wall is located at mid-length of the building, while the second wall is located at the corner near the south face of the building. As shown in Fig. 3.3.2, diagonal shear cracking occurred in the central walls between the first and second floor. This cracking could also be observed on the inside faces of these walls.



Figure 3.3.2 Diagonal Cracking in Central Shear Walls of General Administration Building in Olympia.

Similar diagonal cracking was also observed in the corner walls of the building, as shown in Fig. 3.3.3. Furthermore, the damage observed in the south bottom corners of these walls, as shown in Fig. 3.3.4, suggested also that significant rocking had occurred.



Figure 3.3.3 Diagonal Cracking in Corner Shear Walls of General Administration Building in Olympia.



Figure 3.3.4 Damage in North Bottom Corners of Shear Walls of General Administration Building in Olympia.

Large cracks were also observed in the second floor slab through the ceiling of a large auditorium located in the first story of the building.

Figures 3.3.5 and 3.3.6 show the damage suffered by a concrete detached garage unit located on Franklin Street in Olympia. The concrete frame in the open front suffered cracking in the beam-to-column connections. One existing horizontal crack expanded during the earthquake, causing the column to lean sideways, thereby jamming the garage door.



Figure 3.3.5 Concrete Garage Unit on Franklin Street in Olympia.



Figure 3.3.6 Cracking in Beam-to-Column Joints of a Concrete Garage Unit on Franklin Street in Olympia.

4. PERFORMANCE OF BRIDGE STRUCTURES

With respect to the Nisqually earthquake, bridge structures in the Puget Sound region performed very well. Very few bridges needed to be closed for reasons other than safety inspections and bridges that WSDOT were concerned with, such as the Alaskan Way Viaduct, remained relatively intact. This can largely be attributed to retrofit efforts put forth by WSDOT. Although overall bridge performance was quite good, four bridges that sustained moderate damage are discussed in the following sections.

4.1 Holgate Bridge

The Holgate Bridge is a reinforced concrete overpass connecting the east and west portions of Holgate Street over the Interstate 5 Freeway, as shown in Fig. 4.1.1. Although a retrofit has already been designed by WSDOT for this bridge that was built in 1966, it has not yet been implemented. An aerial perspective would show an initially straight portion on the west end that begins to curve over Interstate 5 and eventually heads south on the east end. The bridge inclines from the west approach leading to an approximate 23 m increase in elevation from the west to the east end.

The bridge substructure consists of eight 1200 mm diameter single column piers of increasing height and an abutment on each end. Approximately 30 No. 14 reinforcing bars with 75 mm concrete cover form the longitudinal reinforcement. No. 4 shear hoops at 300 mm on center are used for transverse reinforcement. The superstructure is a 9100 mm wide concrete box girder with 910 mm overhangs on each side. It serves two lanes of traffic and has a pedestrian sidewalk on the north side.



Figure 4.1.1 General View of Holgate Bridge.

An illustration of the damage that occurred in the westernmost column is presented in Fig. 4.1.2. As shown by the diagonal cracks and the spalled concrete, a substantial shear force developed in this column. The location of the damage relative to the column implies that longitudinal bridge motion was the major contributor to the damage. Evidence of longitudinal displacement is given by fresh scrape marks at a joint in the handrail, as shown in Fig. 4.1.3. Furthermore, the south roller at the west end of the bridge is permanently displaced from its intended relaxed position, as shown in Fig. 4.1.4



Figure 4.1.2 Cracking and Spalling of Concrete in Holgate Bridge Column.



Figure 4.1.3 Slipping of Handrail on Holgate Bridge.



Figure 4.1.4 Displaced Roller on West End of Holgate Bridge.

Evidence of transverse motion given by cracking in the abutment is shown in Fig. 4.1.5. Although the abutment damage is minor, it is apparent that the superstructure impacted the abutment at some instance during the earthquake.



Figure 4.1.5 Cracked Abutment on East End of Holgate Bridge.

4.2 Spokane St. Overcross (SR 99)

The Spokane St. Overcross on State Route 99, constructed in 1958, is 945 m long with 43 m spans. The WSDOT retrofit program for the overcross is composed of three stages. The stage 1 retrofit, which was completed in year 2000, included the installation of restrainers and catchers to prevent unseating of the superstructure as well as steel jacketing of the reinforced concrete columns. The stage 2 retrofit, which was scheduled to begin around the time the earthquake occurred, includes steel jacketing of “split” reinforced concrete columns. The stage 3 retrofit deals with outrigger knee joints.

The Overcross from Bent 25 to Bent 28 sustained damage to the superstructure. Figure 4.2.1 shows the Overcross observed from Bent 25. Both Bent 25 and Bent 28 are constructed of reinforced concrete, with a column height of 14.6 m. The reinforced concrete box girder superstructure transitions to the steel girder superstructure between Bent 25 and Bent 28. Figure 4.2.2 shows the transition at Bent 25. Note the split column with a 50 mm expansion gap that starts at a 9.75 m column height. The gap was introduced to allow for thermal movement of the superstructure.



Figure 4.2.1 Spokane Bridge Overcross.



Figure 4.2.2 Bent 25 with Split Reinforced Concrete Columns.

Figure 4.2.3 shows catchers that were installed underneath each steel girder. Lateral movement of the steel superstructure resulted in the steel brackets bolted to the catcher on each side of the steel girder to shear off, as shown in Fig. 4.2.4. The cross-braced diaphragm at the end of steel superstructure provided the load path to transmit the inertia force at the deck level to the support at the bottom flange level. Figure 4.2.5 shows a buckled brace in the end diaphragm, implying a larger seismic response toward the east direction.



Figure 4.2.3 Stage 1 Seismic Retrofit with Catchers and Restrainers in Bent 25.



Figure 4.2.4 Damage to Catchers.



Figure 4.2.5 End Diaphragm with Buckled Braces in Bent 25.

All eight restrainers that were installed sustained damage, as shown in Fig. 4.2.6. The relative movement between the steel and concrete superstructure caused the restrainers to cut into the web of the channel, in the expansion joint, through which the restrainers passed. Thus, the failure of these restrainers was caused by the transverse movement between the steel and concrete superstructures.



Figure 4.2.6 Damage to Restrainers.

Steel Bent 26 and Bent 27, designed to support the gravity load of the superstructure, are located on both sides of Spokane St. Viaduct, which runs perpendicular to the overcross, as shown in Fig. 4.2.7. Each steel bent is composed of three wide flange columns, oriented with the weak-axis bending in the transverse direction. Figure 4.2.8 shows minor yielding (paint flaking) at the base of an exterior column. No damage was observed in these steel bents.



Figure 4.2.7 Bent 26 with Steel Substructures.



Figure 4.2.8 Yielding at Exterior Column Base of Bent 26.

4.3 Spokane St. Viaduct at 4th Avenue

The elevated Spokane St. Viaduct between I5 and SR99 serves four lanes of traffic, two lanes in each direction. The elevated roadway is composed of repeating segments of continuous reinforced concrete girders connected by steel girders at the surface street intersection. The City of Seattle retrofitted the substructure by steel jacketing reinforced concrete columns every few bents, as shown in Fig. 4.3.1. Where the steel jacket extends below the ground to the top of foundation, PVC pipes are used to make room for the column to deflect laterally. Figure 4.3.1 shows evidence of lateral deflection of the column, yet no damage was observed. Cable restrainers used to connect superstructures also remained intact, as shown in Fig. 4.3.2.



Figure 4.3.1 Steel Jacketed Reinforced Concrete Columns.



Figure 4.3.2 Cable Restrainers in Superstructures.

Steel superstructures and substructures were used at the intersections with surface streets. X-braced bents were used as the substructure in the longitudinal direction, as shown in Fig. 4.3.3. No damage was observed in the braced bents, but flaking of paint at gusset connections, indicative of the seismic effect, was evident as shown in Fig. 4.3.4. Flaking at the column base was also observed, as shown in Fig. 4.3.4. Knee-braced bents were used in the transverse direction, which displayed signs of yield as seen in Fig. 4.3.5. Paint flaking in the steel substructure clearly showed that the bridge was shaken in both directions.



Figure 4.3.3 X-braced Steel Substructure.



Figure 4.3.4 Paint Flaking at Column Base and Gusset Connections.



Figure 4.3.5 Paint Flaking at Knee Brace.

4.4 Fourth Avenue Bridge in Olympia

The Fourth Avenue Bridge spans the east and west ends of a small inlet in the southern tip of Puget Sound. It provides two lanes of traffic with pedestrian walkways on each side. Built in 1920, the bridge uses a Luten concrete arch for the main spans with a standard concrete column supported approach span on the west side, as shown in Fig. 4.4.1. The arch is filled with gravel so that the uniform compression desirable for arched structures can be maintained. The bridge was originally designed for trolley car loads of 7.18 kN/m^2 , which resulted in ample reserve strength for vehicular traffic. However, the bridge is currently in a state of decay, which is depleting this reserve strength. Due to corrosion caused by water seeping through the expansion joints and moderate damage caused by prior earthquakes in the region in 1949 and 1965, the bridge was slated for replacement by the city of Olympia before the

Nisqually earthquake. The bridge is currently closed and traffic is being re-routed to an adjacent bridge.



Figure 4.4.1 Global View of Fourth Avenue Bridge.

Seismic damage to the Fourth Avenue Bridge is isolated to the column supported section and the pedestrian handrails. Square reinforced concrete columns flared on top are used to support the west approach span. Each bent consists of three columns with the flare running parallel to the bridge in the outer columns and perpendicular to the bridge in the inner columns, as shown in Fig. 4.4.2. An example of typical damage that occurred at the base of the column flares is given in Fig. 4.4.3. Spalling of the concrete implies that considerable hinging took place at this location. It is important to note, however, that the bridge was in poor condition prior to the Nisqually earthquake and existing damage was further exacerbated by this recent event. An example of column cracking that existed in 1994 is shown in Fig. 4.4.4.



Figure 4.4.2 Bridge Bent Configuration.



Figure 4.4.3 Concrete Spalling at the Base of Column Flare.



Figure 4.4.4 Bridge Column Condition in 1994².

Also, the pedestrian handrails used on the outer edge of the sidewalks suffered substantial damage, as shown in Fig. 4.4.5. The lack of reinforcement coupled with existing decay made these handrails susceptible to failure during the earthquake.

² AGA. (1994) "Engineering Evaluation of the Fourth Avenue Bridge," Report to the City of Olympia, Arvid Grant Associates Inc., Olympia, WA.



Figure 4.4.5 Failure of a Section of Pedestrian Handrail.

A nearby soil failure resulting from the Nisqually earthquake should be mentioned as it provides some insight into the subsurface composition in the area. Figure 4.4.6 provides an illustration of a slope failure that occurred approximately two blocks from the Fourth Avenue Bridge on the shores of Capitol Lake.



Figure 4.4.6 Soil Failure along the Shore of Capitol Lake.

5. PERFORMANCE OF NON-STRUCTURAL COMPONENTS

From the observations made during the field reconnaissance, it appeared that a large portion of the reported loss associated with the Nisqually earthquake was related to the failure of non-structural components. Although buildings generally behaved well during the earthquake, the performance of non-structural components did not seem consistent with the observed structural performance, thereby reducing the overall performance level of many building systems.

5.1 Performance of Ceiling Systems

One of the most common types of non-structural component failure observed during the field reconnaissance was related to suspended ceiling systems. Examples of partial suspended ceiling failure at Sea-Tac Airport are shown in Figs. 5.1.1 and 5.1.2.



Figure 5.1.1 Partial Failure of a Suspended Ceiling at Sea-Tac Airport.



Figure 5.1.2 Partial Failure of a Metal Suspended Ceiling at Sea-Tac Airport.

One of the buildings that experienced the most damage related to suspended ceiling light fixtures was the Starbucks Headquarters building in downtown Seattle. Although the eccentrically braced steel frames used to seismically upgrade the building performed as intended, the suspending lighting fixtures were unable to accommodate the induced lateral acceleration and caused significant damage as shown in Fig. 5.1.3. Fortunately, only minor injuries resulted from the failure of suspended lighting fixtures throughout the building. The building also suffered damage caused by the shifting and tumbling of unanchored furniture items and building contents, as shown in Fig. 5.1.4.



Figure 5.1.3 Failure of Suspended Lighting Fixtures in Starbucks Headquarters in Seattle.



Figure 5.1.4 Damage Caused by Unanchored Furniture Items and Building Contents in Starbucks Headquarters in Seattle.

5.2 Performance of Interior Wall Finishes

Cracking of interior wall finish materials was observed in many buildings during the field reconnaissance. In most cases, diagonal cracking occurred at upper corners of doors and window openings and at the intersection of beams and walls, as shown in Fig. 5.2.1.



Figure 5.2.1 Cracking of Drywall Finish in a Beam-to-Wall Connection at SeaTac Airport.

One interesting observation on the cracking of drywall finish was made at the Kent Regional District Center. Vertical cracking occurred at the upper corner of almost all interior doors of the building. As shown in Fig 5.2.2, cracks were observed on only one side of each door.



Figure 5.2.2 Vertical Cracking of Drywall Finish Above a Door Opening at Kent Regional District Center.

Plaster spalled from the walls and ceilings of the Legislative Building in Olympia, as illustrated in Fig. 5.2.3. This spalling of plaster above the domed rotunda was one of the concerns that contributed to the closing of the Legislative Building after the earthquake. The building remained closed at the time of writing.



Figure 5.2.3 Spalling of Plaster in Legislative Building in Olympia.

Vertical cracking of wall finishes at the corners of perpendicular walls was observed in a number of buildings. Figure 5.2.4 shows an example of this type of cracking that occurred for the plastered walls of the Supreme Court located in the Temple of Justice Building in Olympia.



Figure 5.2.4 Vertical Cracking of Plastered Walls in the Supreme Court of the Temple of Justice in Olympia.

Substantial cracking of interior wall finish materials was observed in the stair well of the yellow-tagged Olympian Apartments at 519 Washington Street in Olympia.



Figure 5.2.5 Cracking of Plastered Walls in the Stair Wells of the Olympian Apartments in Olympia.

Figure 5.2.6 shows the walls of the original masonry stairwells of the Starbucks Headquarters building in downtown Seattle that experienced severe cracking as a result of the drift level experienced by the building in the east-west direction.



Figure 5.2.6 Severe Cracking of Masonry Stairwells in Starbucks Headquarters in Seattle.

5.3 Performance of Exterior Wall Finishes

Cracking of exterior wall finish materials was observed in residential wood buildings in Olympia. This cracking was usually diagonal and occurred mainly at the corners of window and door openings, as illustrated in Fig. 5.3.1 for a three-story apartment building on Columbia Street in Olympia.



Figure 5.3.1 Diagonal Cracking of Stucco between Window Openings in a Three-Story Apartment Building in Olympia.

In some more recent wood frame houses, wood siding was also damaged as a result of the shaking. Figure 5.3.2 shows an example of a two-story wood frame house on Ninth Avenue in

Olympia that lost some straight wood siding during the earthquake. Note that the lateral movement of the house caused some boards to be wedged against the rafters.



Figure 5.3.2 Damage to the Siding Boards on a Wood Frame House in Olympia.

5.4 Performance of Window Systems

Shattering of glass windows occurred at several locations. The most dramatic instance of this was the loss of all but one window of the control tower at Sea-Tac airport, as shown in Fig. 5.4.1. The failures of these windows contributed to the shut down of the airport for 4 hours following the earthquake as the air-traffic control had to be relocated into a temporary trailer.



Figure 5.4.1 Control Tower Shattered Windows Boarded with Plywood at Sea-Tac Airport.

Boarded windows frames were frequently observed in the Pioneer Square area of Seattle. Workers were commonly seen replacing windowpanes in buildings along the streets, as shown in Fig. 5.4.2.



Figure 5.4.2 Replacing Broken Windowpanes in the Pioneer Square Area of Seattle.

5.5 Performance of Building Contents

Another source of damage to non-structural components was the tumbling of building contents during the earthquake. Figure 5.5.1 shows, for example, the failure of bookshelves that caused some books to fall in the main library of the Temple of Justice Building in Olympia. Figure 5.5.2 shows a detail of the screwed wood-to-metal connection that failed at the top of one of the leaning bookshelves.



Figure 5.5.1 Failed Bookshelves in the Main Library of the Temple of Justice Building in Olympia.



Figure 5.5.2 Failed Screwed Wood-to-Metal Connection at Top of a Leaning Bookshelf in the Main Library of the Temple of Justice Building in Olympia.

This library contained also sturdier movable compact bookshelves mounted on a floor railing. As shown in Fig. 5.5.3, these shelves did not suffer any visible damage during the earthquake and seemed to remain functional.



Figure 5.5.3 Undamaged Movable Compact Bookshelves and Leaning Conventional Bookshelves in the Main Library of the Temple of Justice Building in Olympia.

Although the leaning bookshelves represented an obvious hazard to occupants following the earthquake, only a small number of books fell off these shelves. This is in contrast with what

happened in the Law Library of the same Temple of Justice Building. As shown in Fig. 5.5.4, the massive wood bookshelves of the Law Library did not collapse. A large number of books, however, were thrown to the floor by the horizontal acceleration induced by the ground motion.

Several book shelves and one unanchored computer also toppled over at the Kent Regional District Center.



Figure 5.5.4 Fallen Books from Massive Wooden Shelves in the Law Library of the Temple of Justice Building in Olympia.

The majority of the furniture in the top three floors of the eight-story Ramada Inn at 621 Capitol Way in Olympia was overturned as a result of the building shaking. Figure 5.5.5 shows typical examples of the damage to furniture in quest rooms on the eighth floor. In the bottom five floors most of the furniture remained in the upright position. None of the room furniture in the building had been fastened to the walls to prevent overturning.



Figure 5.5.5 Overturned Furniture in the Guest Rooms on the Eighth Floor of the Ramada Inn in Olympia.

5.6 Performance of Gas and Water Systems

Only one reported fire erupted at the Cedar Creek Correction Center following the earthquake. One gas shut-off valve was activated at the Kent Regional District Center. It was reported that the residents of 50 mobile homes in Tumwater Mobile Estates were evacuated when a 30 mm gas line ruptured during the earthquake.

Several water lines were severed during the ground shaking. One water line and one chilled water line failed on the fourth floor of the Kent Regional District Center.

A 75 mm diameter water pipe broke in the mechanical room on the roof of the Ramada Inn at 621 Capitol Way in Olympia, causing 3000 liters of water in a storage tank to flood several floors of the building. During the earthquake the unsecured water tank was reported to have shimmied about 150 mm along the floor. This, in turn, caused the supply water line to the tank to rupture as shown in Fig. 5.6.1.



Figure 5.6.1 Rupture of the Supply Water Line to a 3000 Liter Storage Tank in the Roof Top Mechanical Room of the Ramada Inn in Olympia.

5.7 Performance of Electrical Systems

No damage to electrical power generation and distribution systems was reported during the period of the field reconnaissance. The only observed effect of the earthquake on electrical equipment was the permanent lateral deformation of a wood pylon in the epicentral region of Nisqually Valley, as shown in Fig. 5.7.1.



Figure 5.7.1 Permanent Lateral Drift of a Wood Electrical Pylon in the Epicentral Region of Nisqually Valley.

6. CONCLUSIONS

The moment magnitude $M_w = 6.8$ Nisqually earthquake that occurred at 10:54 am (PST) on February 28, 2001 caused light-to-moderate ground shaking in the Puget Sound area of Washington State. The focal depth of the earthquake (52 km) contributed in limiting the ground shaking intensity and associated structural damage, even though the strong motion duration of the earthquake was relatively long. It is anticipated that a large portion of the estimated two billion dollar loss resulting from the Nisqually earthquake will be associated to damage to non-structural components. Consequently, even though building structures generally performed well during the earthquake, the inferior performance of non-structural components reduced the overall performance of many building systems.

The effort undertaken to date in Washington State to seismically retrofit older buildings and bridges seemed to be effective in limiting the structural damage produced by the Nisqually earthquake. Without these retrofit procedures it is possible that a number of structures could have failed under the induced ground shaking. It is also to be noted that numerous buildings that were not seismically retrofitted performed well during the earthquake. Nevertheless, considering that earthquakes of higher intensity can occur in this region, authorities and owners should not get a false sense of confidence from the Nisqually earthquake experience, but should continue to seismically retrofit their built environment.