US Army Corps
of Engineers
Portland District

Seismic Performance of the Columbia River Levee Along NE Marine Drive, Portland, Oregon

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ABBREVIATIONS AND ACRONYMS

cm	centimeter(s)
Corps	U.S. Army Corps of Engineers, Portland District
CPT	cone penetration test
MCDD	Multnomah County Drainage District No. 1
M_L	Richter magnitude
M_W	moment magnitude
NGVD	National Geodetic Vertical Datum
OSU	Oregon State University
PGA	peak ground acceleration
USGS	U.S. Geological Survey

Seismic Performance of the Columbia River Levee Along NE Marine Drive, Portland, Oregon

EXECUTIVE SUMMARY

The U.S. Army Corps of Engineers, Portland District, in affiliation with the Multnomah County Drainage District No. 1, studied the seismic performance of the Columbia River levee along NE Marine Drive. The levee system along Marine Drive is particularly important because of the infrastructure it protects. The study evaluated a "worst case" situation involving concurrent flood and earthquake events to determine the statistical probability of damage or failure of the levee.

A flood and seismic hazard investigation was performed by engineers specialized in soils and earthquake engineering from Oregon State University. The study focused on a two-mile long stretch of levee in the center of the drainage district. The objectives included performing a static (hydrologic) stability analysis for the levee, and a seismic stability analysis for the levee at three different river levels including the potential extent of earthquake-induced deformations.

The static stability analysis showed that the levee was stable with excess capacity for a river level at elevation 29.2 feet NGVD (1% flood risk or the 100-year flood event); the levee was found to be unstable for a river level at elevation 42.2 feet NGVD (top of levee). Based on the seismic analysis for the 1% flood risk, the levee is considered safe for a 0.2% seismic risk (M_w 6.2) as well as for a 0.04% seismic risk (M_w 7.0).

The Corps performed an assessment of the risk of interior flooding due to damage of the earthen levee from concurrent hydrologic and seismic events. The combined risk of a seismic event occurring during a major flood event on the Columbia River is very low. A seismic event by itself would not result in interior flooding unless a major flood event was in progress. The risk of these two events occurring at the same time was computed by multiplying the probability of the flood event by the probability of the seismic event. The following risk scenarios were computed in this manner.

- A 100-year flood and a magnitude 6.2 earthquake causing no significant levee damage yields an annual probability of 0.00002 (0.002% risk), or a 1 in 50,000 year chance.
- A 100-year flood and a magnitude 7.0 earthquake causing damage but not levee failure yields an annual probability of 0.000004 (0.0004% risk), or a 1 in 250,000 year chance.
- A 400-year flood and a magnitude 6.2 earthquake causing damage but not levee failure yields an annual probability of 0.000005 (0.0005% risk), or a 1 in 200,000 year chance.
- A 400-year flood and a magnitude 7.0 earthquake causing significant levee damage yields an annual probability of 0.000001 (0.0001% risk), or a 1 in 1,000,000 year chance.

Seismic Performance of the Columbia River Levee Along NE Marine Drive, Portland, Oregon

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Stability Analyses of the Columbia River Levee Adjacent to the Portland International Airport. October 2000. Prepared by S.E. Dickenson, J. Sunitsakul and B.J. Wavra, Department of Civil, Construction, and Environmental Engineering, Oregon State University. Prepared for the U.S. Army Corps of Engineers, Portland District.

1. INTRODUCTION

The U.S. Army Corps of Engineers, Portland District (Corps), in affiliation with the Multnomah County Drainage District No. 1 (MCDD), studied the seismic performance of the Columbia River levee along NE Marine Drive (Figure 1). The MCDD maintains about 13 miles of levee along the Columbia River. The Columbia River levee system along NE Marine Drive is particularly important because of the infrastructure it protects, including the airport, Interstate Highway 205, and many major roadways, municipal water pumping stations, treated wastewater outflow conduits, power and telecommunication lines, businesses and homes. The levee's waterfront portion is used for recreational facilities and provides access to commercial maritime facilities along the river. Damage or failure of the levee during a concurrent flood event and an earthquake could result in flooding of extensive infrastructure protected by the levee.

The seismic hazard in the Portland area reflects the contributions of three seismic sources. The maximum credible earthquake events associated with these sources range in magnitude from 6.5 to 9.0. Recent seismic hazard studies indicate that significant levels of bedrock shaking could occur in the area of the drainage district. These strong ground motions may be capable of initiating liquefaction in the sandy soils that are predominant in the levee and its foundation, which could lead to significant damage or failure of the levee.

The consequences resulting from levee damage or failure highlighted the need for a study of its overall stability when subjected to the flood and seismic hazards present in the Portland metropolitan area. The flood and seismic hazard investigation was performed by Steve Dickenson, Bryan Wavra, and Jutha Sunitsakul from the Department of Civil, Construction, and Environmental Engineering at Oregon State University (OSU). These engineers are specialized in soils and earthquake engineering. This investigation evaluated a two-mile long stretch of levee at the center of the drainage district near the Portland International Airport (Figure 2) that is representative of overall levee conditions. Six sections were selected based on existing levee geometry, soil conditions, and levee accessibility.

A risk assessment was performed by the Corps to determine the level of risk from concurrent flooding and seismic events causing flood damage to the infrastructure protected by the levee (interior flooding). For this report, risk is expressed as a percentage of the annual chance of occurrence. For example, the 100-year flood event is expressed as a 1% flood risk (0.01 probability of occurrence), and the 500-year flood event as a 0.2% flood risk (0.002 probability of occurrence). Similarly, a magnitude 6.2 seismic event is expressed as a 0.2% seismic risk (0.002 probability of occurrence) and a magnitude 7.0 seismic event as a 0.04% seismic risk (0.0004 probability of occurrence).

Figure 1. Site Location

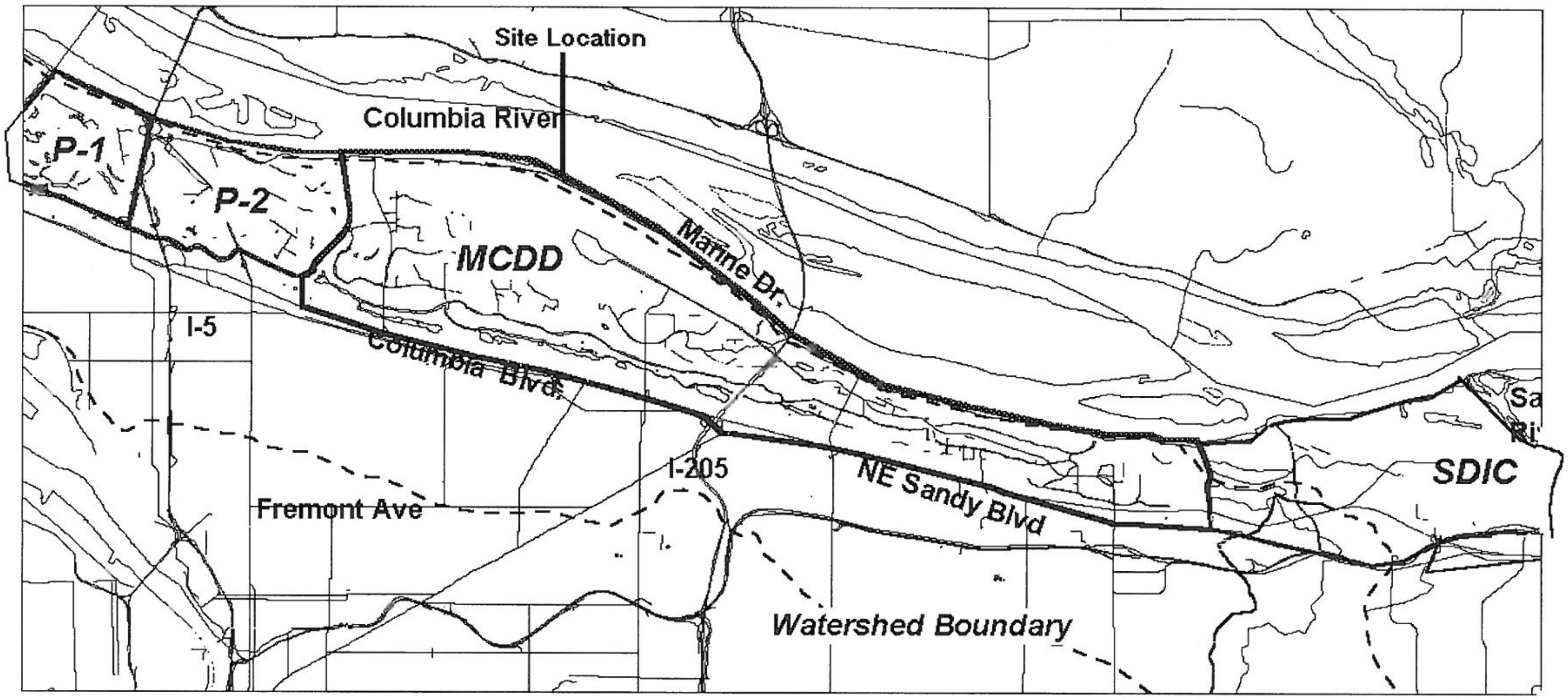
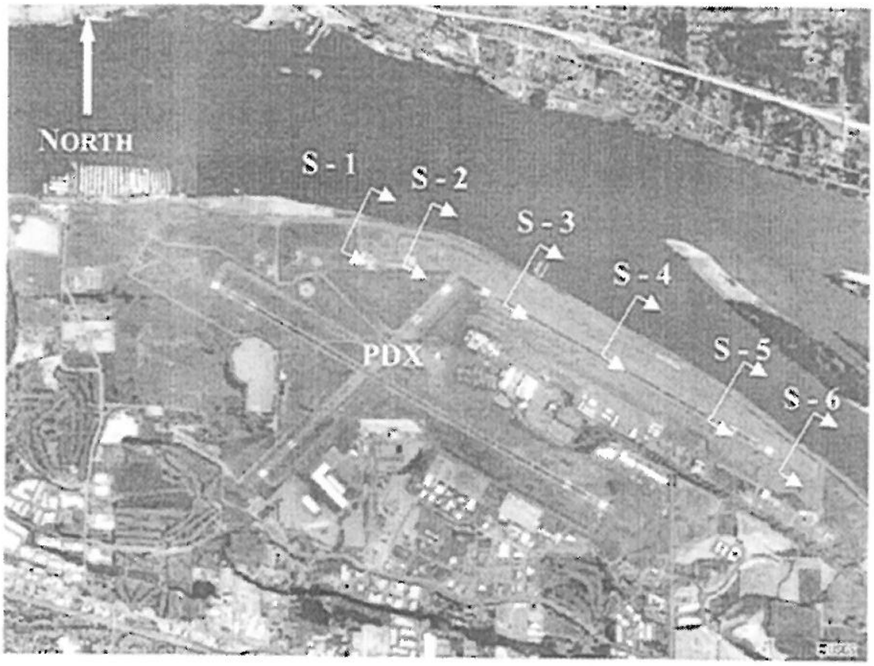


Figure Key:
P-1: Peninsula Drainage District No. 1
P-2: Peninsula Drainage District No. 2
MCDD: Multnomah County Drainage District No. 1
SDIC: Sandy Drainage Improvement Company

Figure 2. Site Location, Levee Sections



*Approximate Scale: ¾ Miles

2. OBJECTIVES OF STUDY

The three primary objectives of this study are shown below.

- Evaluate the seismic stability of the Columbia River levee at three different river levels (OSU).
- Evaluate the potential extent of earthquake-induced deformations of the levee (OSU).
- Perform a risk assessment on the likelihood of concurrent flood and seismic events causing interior flood damage (Corps).

3. BACKGROUND INFORMATION

3.1. Flood Hazard

Since the 1930s, the Federal Government and public and private utilities have constructed numerous dams in the Columbia River Basin for purposes of providing power generation, flood damage reduction, navigation, and irrigation. Those projects that have flood control capability have been effective in maintaining regular flow by storing water during periods of natural high flow and then releasing it during periods of natural low flow.

However, the Columbia experiences two types of flood events, winter floods and spring floods. Spring floods are normally the longest. Both are carefully regulated in an attempt to reduce flood damages. During high water events such as the February 1996 flood, the projects were not capable of regulating the large volume of water entering the basin from snowmelt and rainfall, and the river swelled to unusually high levels.

Levees along the lower Columbia River provide a final barrier of protection against the inundation of approximately 75,000 acres of land, some of which is highly developed. The possible levee problems associated with elevated water levels include the following.

- An increase of the hydraulic gradient within the levee can result in the deformation of the levee, and increases the potential for piping of foundation materials.
- The overall stability of the levee is reduced and slope failures can either occur as a result of the rapid increase in river elevation or in a rapid drawdown situation.
- Levee soils can be eroded due to the increased flow of the river, thereby reducing the effectiveness of the structure.

3.1.1. Multnomah County Drainage District No. 1

The MCDD occupies an area about one mile wide along the Columbia River between river miles 108.2 and 119.0 (Figure 1). The Union Pacific Railroad and U.S. Highway 30 parallel the south boundary. The district is separated from Peninsula Drainage District No. 2 on the west by the Peninsula drainage canal, while the upstream (eastern) end of the district adjoins Sandy Drainage Improvement Company at NE 223rd Avenue.

The MCDD contains 8,417 acres of which Blue Lake, Fairview Lake, Upper Columbia Slough occupies 465 acres along with some other small ponds and sloughs. Land elevations in the district vary between 12 and 25 feet as measured by the National Geodetic Vertical Datum (NGVD).

3.1.2. History of the Columbia River Levee

The MCDD was organized in 1917. During the period from 1919 to 1921, local interests constructed the following flood control system designed to protect the area against a flood equal to that of 1876 (elevation 33.7 NGVD).

- Levees along the Columbia River and Peninsula drainage canal totaling 69,750 feet in length.
- A main pumping plant on Columbia Slough with an installed capacity of 60,000 gallons per minute against a 22-foot head.
- Five tide boxes equipped with wooden gates.
- Five auxiliary pumping plants distributed over the interior of the district.

Sometime before 1939, the Port of Portland and the MCDD raised the riverfront levee to the 1894 flood level (elevation 39.2 feet NGVD) between the west boundary of the airport and the west end of the MCDD, a distance of about 7,000 feet. Between 1939 and 1941, the Corps reconstructed 53,000 feet of levee, using the 1894 flood level as a design criterion, and revetted 4,000 feet of levee along the Columbia River.

Following the flood of 1948 (elevation 36.0 feet NGVD), the Corps restored the damaged levee system by constructing a new pumping plant, replacing the original tide box, and adding sand blanket reinforcement to the levee embankment. Later the Corps increased the effective height of the levee to about one-foot above the 1894 flood level by constructing a parapet dike containing paved roads along the levee grade (Corps 1957).

Since the completion of major Columbia River water storage reservoirs in 1972, the flood elevation has been significantly reduced. Prior to the construction of the Columbia system storage reservoirs, the unregulated 1% flood risk (100-year flood event) was calculated to be at elevation 40.4 feet NGVD as compared to the present regulated elevation of 29.2 feet NGVD with the reservoirs.

In 1998, the drainage district completed a multi-million dollar modernization and capacity expansion of its critical infrastructure. The district's primary pump stations were increased to 600,000 gallons per minute pumping capacity from the Columbia Slough to the river. The modernization included internal equipment upgrades for reliability and redundancy, and a second primary power feeder for electrical power redundancy.

3.1.3. Selection of Flood Elevations for Analyses

Figure 3 shows a typical levee section for the Columbia River levee at NE Marine Drive. Table 1 shows the regulated flood elevations at NE Marine Drive and their level of risk. Table 2 provides the conversion factors between the various elevation data used by agencies in the Portland metropolitan area. Three river elevations were selected for the levee stability and seismic analyses. These elevations represent a broad range of conditions so that the sensitivity of levee stability to varying river levels could be assessed.

- Elevation 7.0 feet NGVD, a low river level representative of summer/fall conditions.
- Elevation 29.2 feet NGVD, the 1% flood risk (100-year flood elevation).
- Elevation 42.4 feet NGVD, representing a "worst-case" scenario with the river level at the levee crest.

Figure 3. Typical Levee Section, Columbia River Levee at NE Marine Drive

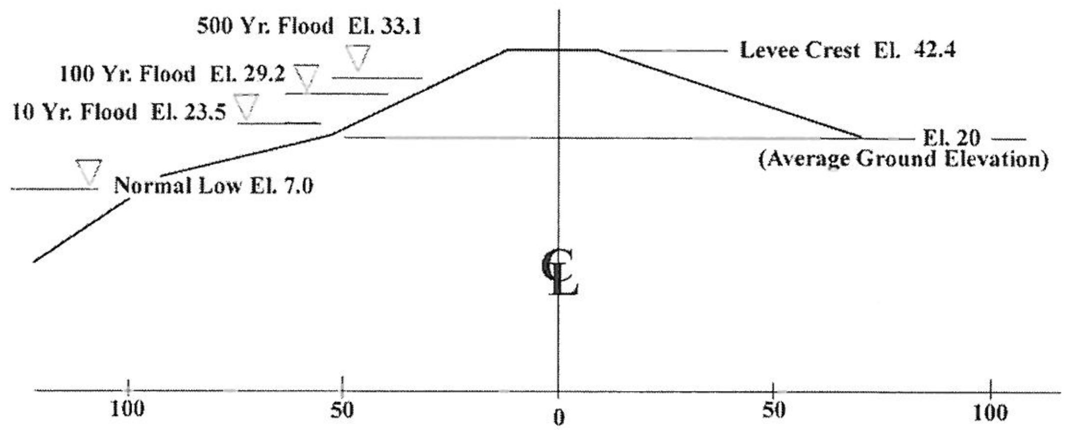


Table 1. Regulated Flood Elevations at NE Marine Drive

Risk	Flood Event	Elevation (feet NGVD)
0.10 (10%)	10-year	23.5
0.02 (2%)	50-year	27.3
0.01 (1%)	100-year	29.2
0.0025 (0.25%)	400-year	32.7
0.002 (0.2%)	500-year	33.1
-----	Levee Design Flood	34.3
-----	Levee Crest	42.4

Table 2. Conversion Table for Elevation Data

	Datum (elevation in feet)				
	NGVD	Mean Sea Level	City of Portland	National Weather Service	NAVD88
Conversion	0.0	0.0	1.4	-1.8	3.5

3.2. Seismic Hazard

The Richter scale is a commonly known measure of earthquake magnitude (M_L) and is based on a logarithmic scale. An increase of one unit of magnitude (for example, from M_L 3.0 to 4.0) represents a 10-fold increase in the extent of ground shaking as recorded on a seismograph. Table 3 provides the effects of earthquakes at various Richter magnitudes.

Table 3. Richter Earthquake Severity

Richter Magnitudes	Earthquake Effects
Less than 3.5	Generally not felt, but recorded
3.5 to 5.4	Often felt, but rarely causes damages
Under 6.0	At most slight damage to well-designed buildings; can cause major damage to poorly constructed buildings over small regions
6.1 to 6.9	Can be destructive in areas up to about 100 kilometers across
7.0 to 7.9	Major earthquake; can cause serious damage over larger areas
8.0 or greater	Great earthquake; can cause serious damage in areas several hundred kilometers across

The Moment-magnitude scale (M_w) is replacing the Richter scale because it gives a more accurate interpretation of the amount of energy released by an earthquake. The amount of energy released is related to rock properties such as rock rigidity, area of the fault surface, and the amount of movement on the fault.

The overall seismic hazard in the Portland metropolitan area reflects the contributions of the following three seismic sources.

- Interplate earthquakes along the Cascadia Subduction Zone, where the oceanic Juan de Fuca plate plunges under the continental North American plate about 60 to 150 miles offshore of the Pacific coast. This 750-mile long fault runs parallel to the coast from British Columbia to northern California (Figure 4).
- Intraslab earthquakes are relatively deep subduction zone earthquakes that may be located as far inland as the Portland area (subduction is the process in which one plate collides with and is forced down under another plate).
- Relatively shallow crustal earthquakes located in the Portland area.

An *interplate earthquake* occurs due to movement at the interface of tectonic plates. These earthquakes are usually relatively shallow thrust events, occurring in the upper 50 kilometers of the earth's crust. The Cascadia Subduction Zone, consisting mainly of the interface of the Juan de Fuca and the North America plates, provides the potential for subduction zone events in the Pacific Northwest. A generalized section of the Cascadia Subduction Zone for Oregon is shown in Figure 5. For the purpose of this study, the eastern edge of the seismogenic portion of the interface was assumed to be about 80 kilometers west of Portland, and about 25 kilometers below mean sea level.

Figure 4. Plate-tectonic Map of the Pacific Northwest

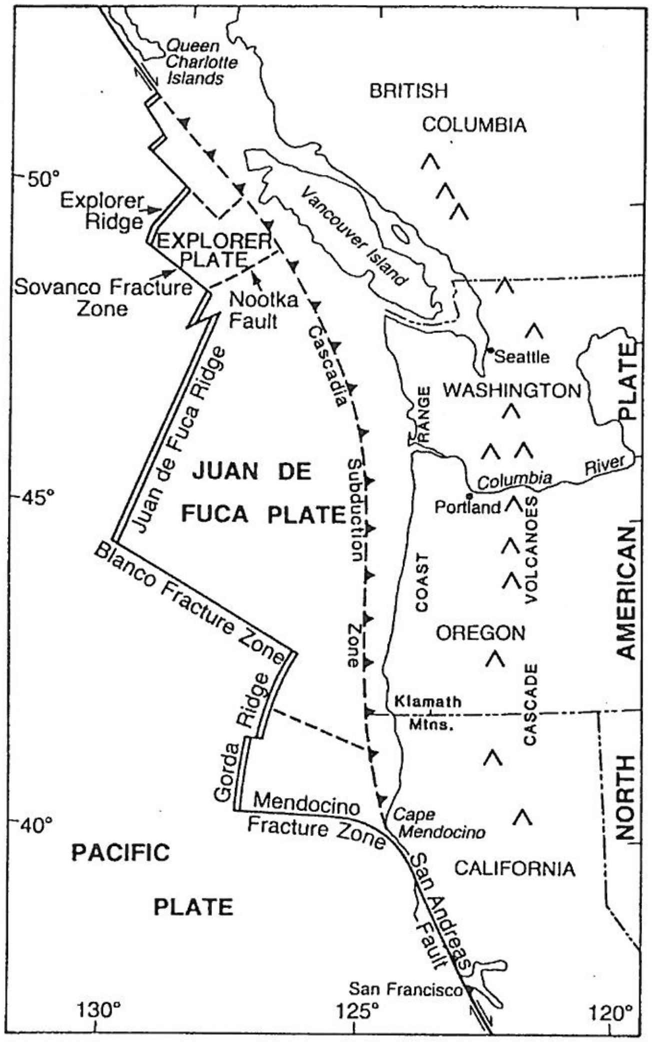
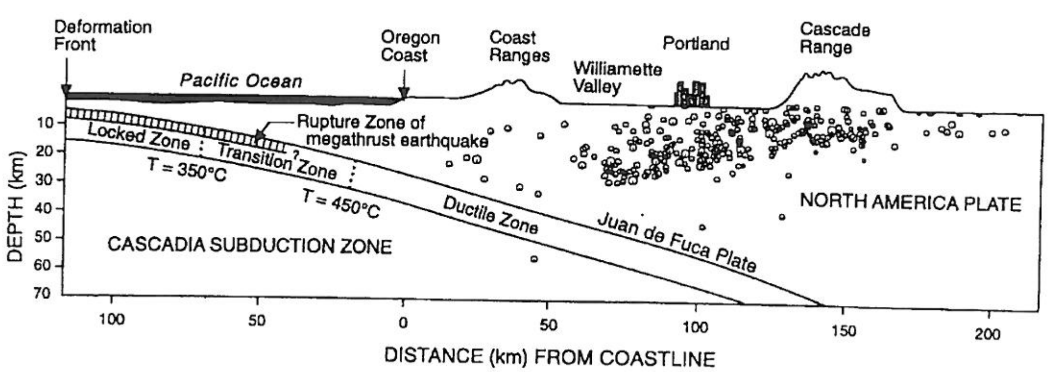


Figure 5. Cascadia Subduction Zone, Generalized Section for Oregon



An *intraslab earthquake* originates within a subducting tectonic plate and occurs at distances from the edges of the plate. It is caused by the release of built up stresses within a tectonic plate as it dives below an overriding plate. For Oregon, the Juan de Fuca plate provides the potential for such an event. The magnitude 7.1 Olympia earthquake of 1949, the magnitude 6.5 Puget Sound earthquake of 1965, and the magnitude 6.8 Olympia earthquake on February 28, 2001 are examples of intraslab earthquakes occurring in the Juan de Fuca plate.

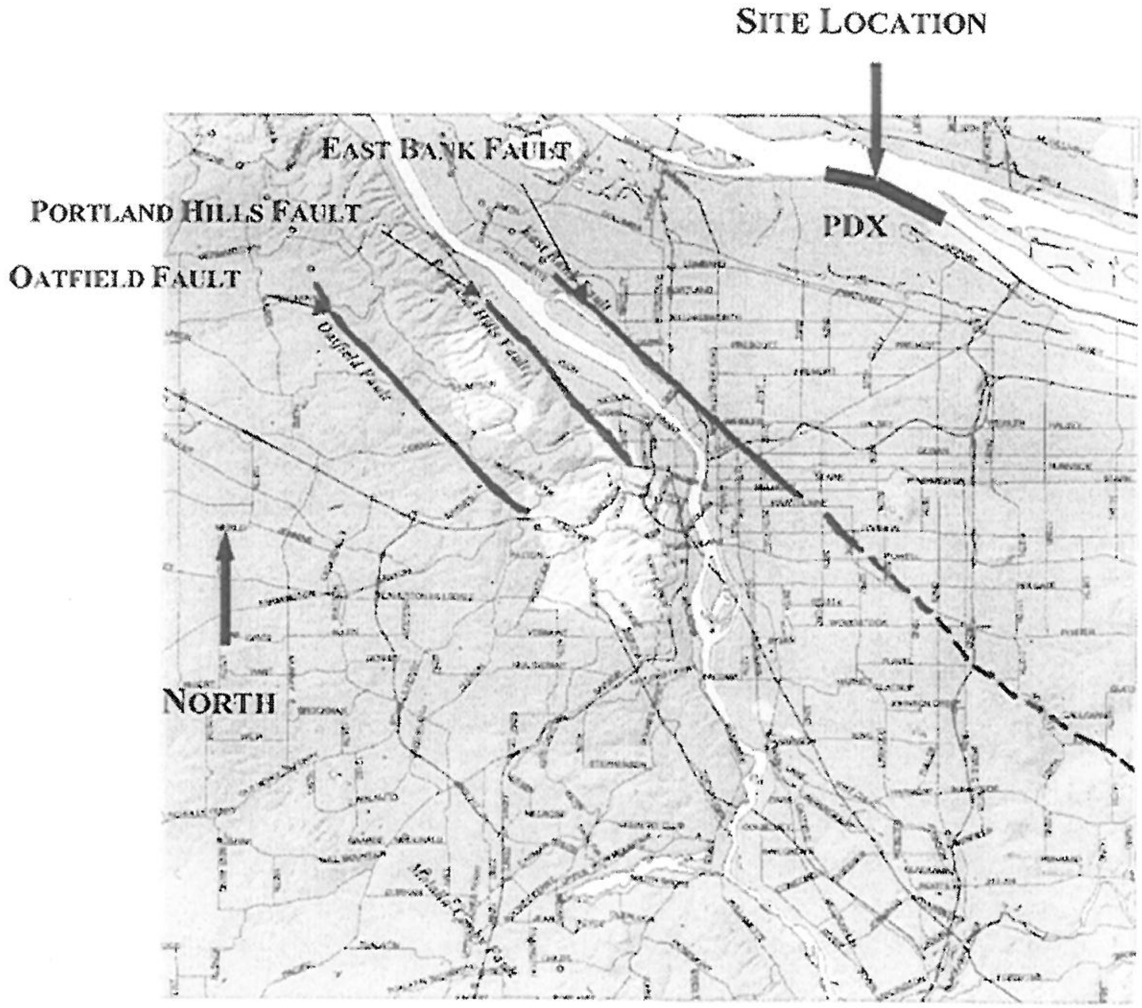
Crustal earthquakes are caused by movements along faults in the upper 20 kilometers of the earth's crust. In Oregon, these movements occur in the crust of the North American Plate when built up stresses near the surface are released. Recent examples of crustal earthquakes in Oregon include the M_L 5.6 Scotts Mills earthquake in 1993 and the M_L 6.0 Klamath Falls earthquake of 1993.

During the last 150 years, the Portland area has been subjected to six earthquakes of M_L 5.0 or greater, including the M_L 5.5 Portland earthquake of 1962 and the Scotts Mills earthquake (Wong et al. 2000). Recent studies have been performed that indicate the presence of three crustal faults beneath the Portland area, which could generate damaging crustal earthquakes of M_L 6.5 or larger (Blakely et al. 1995; Pratt et al. 1999; Wong et al. 2000). These faults are the Oatfield, Portland Hills, and East Bank faults (Figure 6).

Seismic hazard maps for the Portland area showing the underlying faults have been developed (Wong et al. 2000). The East Bank Fault was used as the design fault in this study because of its close proximity to the project site (8 kilometers). In addition, the Cascadia Subduction Zone has been estimated to be capable of generating a M_w 8.0 to 9.0 earthquake (Geomatrix 1995).

According to Wong et al. (2000), an evaluation of earthquake recurrence based on the historical record suggests that crustal earthquakes of M_L 6.5 or larger occur somewhere in the Portland region on average about every 1,000 years. Also, a convincing case has been made to indicate that Cascadia subduction zone earthquakes up to M_w 9.0 have occurred in the prehistoric past as recently as the year 1700, and will occur in the future. Therefore, the Portland metropolitan area has gone relatively unscathed by damaging earthquakes; strong ground shaking generated by a Cascadia subduction zone earthquake or a nearby crustal event will have a major future impact on the area (Wong et al. 2000).

Figure 6. Main Earthquake Faults in the Portland Area



***Approximate Scale:** **3 miles**

4. Geotechnical Investigation

4.1. Methodology

Soil reports from the Corps and the Port of Portland were reviewed along with information from local engineering consulting firms, the Oregon Department of Geology and Mineral Industries, and the technical literature. This information was supplemented with the results of a more rigorous, site-specific investigation with the following objectives.

- Perform field surveys to establish geometric profiles of the Columbia River levee.
- Perform *in situ* tests using cone penetrometer (CPT) and dilatometer soundings (testing devices that provide information on soil parameters) to establish soil layering, shear wave velocities, pore water pressures, and other data for use in the liquefaction analyses and correlations to determine soil properties.
- Perform auger borings to supplement the *in situ* soil profile data and to provide soil samples for use in the laboratory investigation.

Four exploratory mud-rotary borings, 12 CPTs, and 1 dilatometer sounding were taken for the investigation. Two additional CPTs were taken adjacent to the pumping station on the southeast levee at the MCDD office. The samples obtained during the field work were transported to the Geotechnical Engineering Laboratory at OSU, and the following laboratory tests were conducted: moisture/density, gradation, liquid and plastic limits (Atterberg limits), consolidation, direct shear, and compression tests.

4.2. Summary of Results

The MCDD is a diked-off portion of the Columbia River flood plain. The materials underlying the flood plain to about elevation -80 feet are stratified sediments consisting of silts, clays, sands, and blends of these materials. In the western part of the district, water wells penetrate gravel, typically referred to as the Troutdale Formation, at about elevation -80 feet. The contact between the gravel and the fine sediments slopes off sharply to the north and west (Corps 1957). It is estimated that the elevation of the Troutdale Formation is about -180 feet at the project site and is subsequently underlain by Sandy River mudstone and Columbia River basalt (Wong and Silva 1993).

The levee geometry and crest elevation was rather uniform over the project site with the crest elevation varying from 42.9 feet to 43.3 feet NGVD. The riverward slopes have been benched with slopes of approximately 2 horizontal to 1 vertical, while landward slopes were typically 3 horizontal to 1 vertical. The soils encountered at the various levee cross-sections consisted of primarily poorly-graded sand and silty-sand within the levee, with foundation deposits of predominantly silty soils. Sand was encountered between elevation -20 feet and elevation -30 feet throughout the project site.

It is estimated that the sand continues down to the Troutdale Formation at elevation -180 feet, which is subsequently underlain by Sandy River mudstone at approximately -250 feet (Wong and Silva 1993).

5. Ground Shaking Evaluation

5.1. Methodology

Probabilistic uniform hazard studies have been completed in recent years for the Portland metropolitan area, which provide estimates of the peak ground acceleration (PGA) values on rock (Geomatrix 1995; USGS 1996). These hazard studies incorporate the relative contributions of the three seismic sources in the Portland area to develop one PGA value for specified probabilities of exceedance in given time periods. For this report, PGA values are estimated for earthquakes with a 0.2% seismic risk (M_w 6.2; 0.002 probability of occurrence) and a 0.04% seismic risk (M_w 7.0; 0.0004 probability of occurrence).

A major part of the OSU investigation was to produce accurate estimates of expected bedrock ground motions from earthquakes at the project site. These estimates are typically empirically based using actual strong ground motion records. The equations are expressed as a function of both magnitude and source-to-site distance yielding predictions of PGA and acceleration response spectra, termed "target spectra." The target spectra used in this investigation represent an average of the individual spectra produced by three crustal attenuation relationships by Abrahamson and Silva (1997), Boore et al. (1997), and Campbell (1997), while the target spectra for the subduction zone event was generated from one attenuation relationship (Youngs et al. 1997). The attenuation relationships are regression equations and were derived based on a suite of strong motion recordings and have fault rupture type, distance, and site conditions built into them.

The values generated from the attenuation relationships were then used to select recorded time histories of like earthquake events with similar frequency content as the target spectra. The selected time histories were then scaled to the predicted bedrock PGA. An effort was made to match distance, magnitude, spectral content and PGA for the selected time histories with those of the predicted values, though the limited number of recorded time histories did not always allow for complete agreement.

5.2. Summary of Recent Seismic Hazard Investigations

Comparisons were made between the recommendations of the PGA on soil and rock from the following sources: Geomatrix (1995), the U.S. Geological Survey's (USGS) *National Seismic Mapping Project* (1996), and the *Ground Shaking Maps for Portland* (Wong et al. 2000). The investigations by Geomatrix and the USGS provide recommendations for *PGA* on bedrock, whereas Wong et al. (2000) yields PGA values at the soil surface.

These recommendations are based on hazard studies that combined the ground shaking contributions from the interplate, intraplate, and crustal earthquake scenarios into one peak ground acceleration value for the 0.2% (M_w 6.2) and the 0.04% (M_w 7.0) seismic risks (Table 4).

Table 4. Comparison of Peak Ground Acceleration Values for the Project Site

Source	**Ground Motions Associated with Specified Probabilities of Exceedance**			
	PGA_{rock}		**PGA_{soil}**	
	0.2% Risk	0.04% Risk	0.2% Risk	0.04% Risk
USGS	0.19g*	0.38g	N/A	N/A
Geomatrix (1995)	0.19g	0.37g	N/A	N/A
Wong et al. 2000	N/A	N/A	0.20g – 0.25g	0.40g

* "g" is a common value of acceleration equal to 9.8 meters/sec/sec (the acceleration due to gravity at the surface of the earth).

5.3. Subduction Zone Bedrock Motions

Ground motion characteristics were estimated for M_w 8.5 and M_w 9.0 subduction zone earthquakes with a source-to-site distance of 85 kilometers. The M_w 8.5 earthquake was chosen based on the recommendations published by Geomatrix (1995) and the M_w 9.0 was recommended by Wong et al. (2000). The PGA_{rock} was 0.12g and 0.14g for the M_w 8.5 and M_w 9.0 earthquakes, respectively, using an attenuation relationship created by Youngs et al. (1997). Recorded time histories from the M_w 8.5 Michoacan earthquake in 1985 and the M_w 8.0 Miyagi-Oki subduction zone earthquake in 1978 were compared against the predicted target spectra. The selected records provided a satisfactory match of spectral content and they were used as the design input bedrock ground motions for the dynamic ground motion analysis.

5.4. Crustal Bedrock Ground Motions

Attenuation relationships by Abrahamson and Silva (1997), Boore et al. (1997), and Campbell (1997) were used to determine the spectral content of the crustal earthquake motions. The recommended design level earthquake having about a 0.2% risk is M_w 6.2 (Geomatrix, 1995). The M_w 6.2 is consistent with other seismic hazard studies, and it has been specified in this investigation to occur on the East Bank fault. Given the close proximity of the East Bank fault to the levee site, the attenuation relationships yielded PGA_{rock} values that were higher than those recommended by the USGS (1996) and Geomatrix (1995) for a seismic event with a 0.2% risk.

Wong et al. (2000) estimated that the East Bank fault is capable of M_w 6.8 earthquakes. This magnitude was rounded up to M_w 7.0 to represent a seismic event with a 0.04% risk. This assumption was validated by the crustal attenuation relationships using a M_w 7.0 earthquake on the East Bank fault which yielded an average PGA_{rock} value of 0.38g. This also is consistent with the PGA values given by Geomatrix (1995) and the USGS (1996) for a uniform hazard study earthquake with a 0.04% risk.

Numerous acceleration response spectra were evaluated in order to select the time histories that most closely matched the target spectra, PGA, duration, source-to-site distance, and magnitude for the crustal event. Selected records from the M_w 6.2 Mammoth Lakes earthquake in 1980 and the M_w 6.4 San Fernando earthquake in 1971 were determined to best match the M_w 6.2 event, while motions from the M_w 7.0 Northridge earthquake in 1994 and the M_w 7.0 Loma Prieta earthquake in 1989 were chosen for the M_w 7.0 event. These design earthquakes were scaled to PGA_{rock} values of 0.29g and 0.38g, respectively, for the dynamic ground motion analyses.

5.5. Soil Response Analyses

Soil response analyses were conducted for the project area to compute both the PGA values in the soil versus depth, and the cyclic shear stresses induced as a result of strong ground shaking. Local site conditions have a strong influence on ground motions resulting from earthquakes. The computer program SHAKE91 (Idriss and Sun 1992) was used to perform the soil response analyses and to compute the response of the subsurface to vertically propagating seismic (shear) waves. Seismic waves are caused by the release of energy as a fault suddenly slips, and creates most of the destructive effects of earthquakes.

For each acceleration time history, SHAKE91 was used to compute (1) the acceleration response spectrum on rock to compare against the target spectra generated by the attenuation relationships, (2) the PGA at the top of each defined layer to create a profile of PGA versus elevation, (3) equivalent uniform shear stresses which were used in the liquefaction analyses (see Section 7 of this report), and (4) acceleration time histories to be used in the Newmark sliding block method to estimate levee deformation (see Section 8 of this report).

The profiles of PGA versus elevation in the project area show a significant damping effect (energy dissipation) of the PGA values for the crustal events (M_w 6.2 and M_w 7.0 earthquakes). There was an approximate 50% reduction in the PGA_{rock} values when compared to the PGA_{soil} values at the top of the levee. For example, for the crustal M_w 6.2 event, the PGA_{rock} value was found to be 0.29g, whereas the PGA_{soil} value was 0.16g. In contrast, the PGA_{soil} values were found to be very similar to those for PGA_{rock} for the subduction zone events (M_w 8.5 and M_w 9.0 earthquakes).

6. Soil Liquefaction Hazard Analysis

6.1. Methodology

An important type of ground failure that can occur during an earthquake is known as soil liquefaction. Liquefaction takes place when loosely packed, water-logged sediments at or near the ground surface lose their strength in response to strong ground shaking. Liquefaction occurring beneath buildings and other structures can cause major damage during earthquakes. For example, during the Loma Prieta earthquake in California, liquefaction of the soils and debris used to fill in a lagoon caused major sinking, fracturing, and horizontal sliding of the ground surface in the Marina district in San Francisco.

Liquefaction occurs in saturated soils, which occurs when the space between individual soils particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure is relatively low. However, earthquake shaking can cause the water pressure to increase to the point where the soil particles can readily move with respect to each other. When liquefaction occurs, the strength of the soil decreases and the ability of a soil to support buildings and other structures is compromised.

The liquefaction susceptibility of the soils in the levee and its foundation were assessed on the basis of the *in situ* and laboratory data collected. State of practice methods for evaluating the liquefaction and post-liquefaction behavior of the soils were applied for seismic loading conditions representative of the three primary earthquake hazards in the region. This investigation focused on excess pore pressure generation (shows whether a soil may experience liquefaction), residual undrained shear strengths, and volumetric changes of the predominantly silty and sandy deposits.

The seismic load levels considered in the liquefaction hazard analyses included ground motions having a 0.2% (M_w 6.2) and a 0.04% (M_w 7.0) seismic risk. These risk levels are compatible with recent seismic hazard studies for the region. The liquefaction analyses were performed assuming that all soils within the levee were saturated, which is a conservative assumption.

6.2. Summary of Results

The silty soils along the Columbia River levee are susceptible to the generation of excess pore pressure leading to liquefaction. The liquefaction hazard of the levee and its foundation soils were found to be minimal for the M_w 6.2 earthquake. The silty foundation soils of the levee appear to be liquefiable for the M_w 8.5 and M_w 9.0 earthquakes.

7. SEISMIC PERFORMANCE EVALUATION

A primary goal of this investigation was to assess the seismic performance of the levee at various levels of ground shaking and at different river levels. The performance of the levee was evaluated in terms of the lateral and vertical deformations resulting from the ground motions. The potential for lateral spreading and possible flow failures were addressed. The lateral deformations were computed using empirically based methods for estimating lateral deformations and from a representative number of numerical stress analyses. The deformation analyses focused only on the response of the levee and foundation soils.

The levee does not have to experience a significant amount of deformation to fail. Small cracking may create conduits within the levee that could lead to piping of the fine-grained material. Changes in the levee geometry can alter the flow path of water, which could potentially increase seepage forces, and a "quick" condition may result in a catastrophic failure. Piping and "quick" condition scenarios were not directly analyzed within the scope of this investigation. The analyses estimated the potential deformation or failure of the levee under static (hydrologic) and dynamic (seismic) loading conditions.

Maps of lateral spread displacement along the Columbia River immediately west of the study site have been developed by Mabey et al. (1993). The displacements estimated for a magnitude 8.5 earthquake, 100 kilometers away, on silty-sand soils (30% fines) were between 60 to 122 cm (2 to 4 feet; Mabey et al. 1993). Estimated displacements of this magnitude warranted a more in-depth analysis. An initial evaluation for levee stability was performed using commercially available models, SEEP/W and SLOPE/W (GEO-SLOPE International).

7.1. Initial Evaluation for Levee Stability

The stability of the levee is influenced by the seepage conditions and the properties of the materials in the levee and its foundation. During an earthquake, the stability is also influenced by the degree of shaking experienced (acceleration) and the duration of the shaking. Seepage analysis was performed using the SEEP/W model. Seepage conditions were determined for three conditions with the river at elevations 7.0, 29.2 (the 1% flood risk), and 42.4 (levee crest) feet NGVD. Stability analysis was then performed using the SLOPE/W model that uses limit equilibrium theory to solve for the factor of safety of earth structures and slopes. The analysis was run using the seepage conditions generated from SEEP/W using static soil shear strength properties to determine the 'without earthquake' factors of safety for the levee. The analysis was then run using the dynamic soil properties to determine the 'with earthquake' factors of safety.

The analysis showed the levee to have adequate without earthquake stability for all river levels except for water at the crest of the levee. The analysis also showed the levee to be stable to above the 1% flood risk (elevation 29.2 feet NGVD) for the magnitude 6.2 and

7.0 earthquakes; areas of instability on the landward side of the levee were found during the 8.5 and 9.0 magnitude earthquakes.

Those areas found to be unstable were further analyzed to compute the amount of deformation resulting from these higher magnitude earthquake events. The expected deformation of the levee was estimated using three different methods: (1) the Newmark (1965) sliding block method, (2) the Makdisi and Seed (1978) sliding block method, and (3) a numerical model analysis.

7.2. Newmark and Makdisi-Seed Methods to Estimate Levee Deformation

These methods are rigid body, sliding-block analyses that assume that the soil behaves as a rigid, perfectly plastic material. Using the Newmark method, the levee was found to be stable using the *in situ* strengths for all earthquake events except for the worst-case scenario (river level at top of levee) where the levee has a static (hydrologic) factor of safety less than 1.0 for a landward failure. The probability of the levee sustaining significant damage without soil liquefaction is considered remote. The levee is more susceptible to failure if the soils liquefy. However, large deformations would not be expected unless the river level is at or above the 1% flood risk elevation (29.2 feet NGVD) *and* an earthquake of M_w 8.5 or greater occurs at the same time. Tables 5 and 6 show the deformation estimates using the Newmark method for levee Section 4 for the magnitude 6.2 and magnitude 8.5 earthquakes, respectively.

Table 5. Deformation Estimates at Levee Section 4 for Magnitude 6.2 Earthquake

Earthquake Magnitude	River Level	Landward Movement (cm)			Riverward Movement (cm)		
		Newmark	Makdisi-Seed	Numerical Model	Newmark	Makdisi-Seed	Numerical Model
6.2	El. 7 feet	0	0	0	0	0	8
	1% Flood Risk	0	0	6	0	0	49
	Top of Levee	N/A	N/A	50	0	0	56

1 inch = 2.54 centimeters (cm)

Table 6. Deformation Estimates at Levee Section 4 for Magnitude 8.5 Earthquake

Earthquake Magnitude	River Level	Landward Movement (cm)			Riverward Movement (cm)		
		Newmark	Makdisi-Seed	Numerical Model	Newmark	Makdisi-Seed	Numerical Model
8.5	El. 7 feet	0	0	0	12	500	4
	1% Flood Risk	N/A	N/A	2	< 5	120	43
	Top of Levee	N/A	N/A	38	0	0	53

1 inch = 2.54 centimeters (cm)

The results using the Makdisi-Seed method (Table 6) were generally consistent with those from the Newmark method except for a couple of instances. The Makdisi-Seed method predicts significant displacements for the M_w 8.5 and M_w 9.0 earthquakes for the landward failure mass using static strength values when the river level is at the 1% flood risk elevation (29.2 feet NGVD). A discrepancy also exists where Makdisi-Seed predicts larger deformations for the M_w 8.5 and M_w 9.0 earthquakes for the riverward failure mass using residual strength values. Estimates from the Makdisi-Seed analysis resulted in values that were considered highly over conservative. Both the Newmark and Makdisi-Seed methods are subject to the same limitations in that deformation estimates cannot be made if the static factors of safety are less than one. Also, estimated displacements that are greater than about 100 cm (3.3 feet) are approximate.

7.3. Numerical Model Analysis to Estimate Levee Deformation

A total of six numerical model analyses were performed on the levee. Time histories from the Mammoth Lakes (M_w 6.2) and Michoacan (M_w 8.5) earthquakes were applied to the levee with the river stage at the three design water levels. The deformation estimates using the numerical model for levee Section 4 are shown in Table 6. Maximum horizontal and vertical displacements were less than two feet, which is consistent with estimates by Mabey et al. (1993).

Two conclusions can be drawn from the results of the numerical modeling. First, the deformation of the levee gets progressively larger as the river level increases. As the river level increases, the level at which voids in the soil are filled with water also rises in the levee resulting in the saturation of more soil. These saturated soils become susceptible to liquefaction, which leads to larger deformations of the levee. Second, the deformations induced by the Mammoth Lakes and Michoacan time histories are essentially the same for each river level even though their respective PGA and duration values are significantly different. Therefore, one could expect to see approximately the same amount of deformation for the crustal and subduction zone earthquakes.

8. RISK ASSESSMENT

8.1. Overview

The Corps performed a risk assessment for the Columbia River levee at NE Marine Drive that evaluated the risk of interior flooding due to damage of the earthen levee from concurrent hydrologic and seismic impacts. Factors used for the risk assessment included past levee performance, existing hydrologic data, existing design data, and the static (hydrologic) and seismic analysis performed by OSU for the levee.

8.2. Past Levee Performance

Since 1941, the levee design has provided flood protection from river levels up to elevation 39.2 feet NGVD. Since that time, the levee has been subjected to several flood events, the highest occurring in the spring of 1948 when the river peaked at elevation 36.0 feet NGVD. Between 1948 and 1972 when the last of the major storage reservoirs were completed, the river level at the Multnomah Drainage District No. 1 has been at or above the present regulated 1% flood risk five times (100-year flood event at 29.2 feet NGVD or the 0.01 probability flood). The levee has performed well when subjected to these floods including the 1948 event. In 1978, a safe levee height was set at elevation 31.1 feet NGVD at the downstream end, 34.3 feet NGVD at the upstream end, and about elevation 32.7 NGVD at the midpoint. This is approximately 2.0 feet above the 1% flood risk (0.01 probability flood) and about 0.5 feet below the 0.2% flood risk (0.002 probability flood or the 500-year flood event). This would establish the hydrologic risk for the levee at less than 0.25% (0.0025 probability flood), which is equivalent to about a 400-year flood or greater.

8.3. Static Stability

The static (hydrologic) stability was computed for the levee at river elevations 7 feet, 29.2 feet (1% flood risk), and 42.4 feet NGVD (top of levee). The static stability analysis showed that the levee was stable with excess capacity for a river level at elevation 29.2 feet NGVD (1% flood risk or the 100-year flood event); the levee was found to be unstable for a river level at elevation 42.2 feet NGVD (top of levee). These findings were consistent with previous work, which found the risk of levee failure near zero for a river level at elevation 32.7 feet NGVD and the risk of levee failure near 100 percent for a river level near the top of the levee.

8.4. Seismic Stability

The seismic stability of the levee is discussed in Sections 5 through 7 of this report. As expected, the degree of calculated damage to the levee due to a seismic event increased with the depth of the water on the levee and with the severity of the seismic event. Levee embankment stability and levee deformations due to liquefaction were computed for a 1% flood risk (elevation 29.2 feet NGVD). Factors of safety remained above 1.9 (very stable) for all cases except for the M_w 9.0 earthquake. Deformations remained within an acceptable range for a river level at elevation 29.2 feet NGVD and all earthquake magnitudes except for a M_w 9.0 earthquake. Lateral spreading was limited to non-critical sections of the levee. Based upon the seismic evaluation for a river level at elevation 29.2 feet NGVD (1% flood risk), the levee is considered safe for the 0.2% seismic risk (M_w 6.2) as well as for the 0.04% seismic risk (M_w 7.0), but with some noticeable settlement.

8.5. Combined Flood and Seismic Risk

The combined risk of a seismic event occurring during a major flood event on the Columbia River is very low. There is no known correlation between high water periods and the occurrence of earthquakes. A seismic event by itself would not result in interior flooding unless a major flood event was in progress. The risk of these two events occurring at the same time can be computed by multiplying the probability of the flood event by the probability of the seismic event. The following risk scenarios were computed in this manner.

- A 100-year flood and a magnitude 6.2 earthquake that causes no significant levee damage yields an annual probability of 0.00002 (0.002% risk), or a 1 in 50,000 year chance (0.01 flood probability x 0.002 seismic probability = 0.00002).

- A 100-year flood and a magnitude 7.0 earthquake that causes damage but not failure of the levee yields an annual probability of 0.000004 (0.0004% risk), or a 1 in 250,000 year chance (0.01 flood probability x 0.0004 seismic probability = 0.000004).

- A 400-year flood and a magnitude 6.2 earthquake that causes damage but not failure of the levee yields an annual probability of 0.000005 (0.0005% risk), or a 1 in 200,000 year chance (0.0025 flood probability x 0.002 seismic probability = 0.000005).

- A 400-year flood and a magnitude 7.0 earthquake that causes significant levee damage yields an annual probability of 0.000001 (0.0001% risk), or a 1 in 1,000,000 year chance (0.0025 flood probability x 0.0004 seismic probability = 0.000001).

The damages assumed for the 400-year flood (elevation 32.7 feet NGVD) were based upon interpolation of factors of safety and deformation computed at elevation 29.2 feet NGVD (1% flood risk) and those determined for water levels at the top of the levee (elevation 42.4 feet NGVD).

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