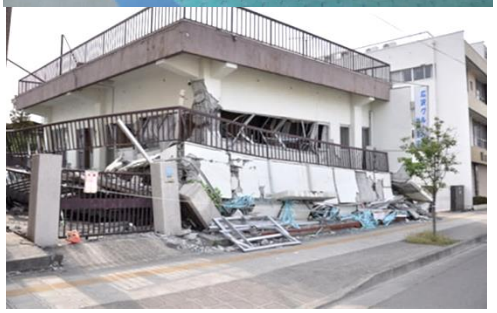

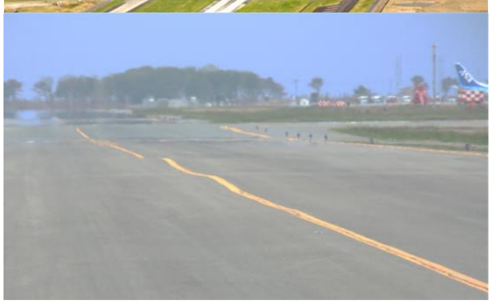

Port of Portland Corporate Seismic Risk Assessment Study

Final Report

May 2015

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PREFACE

Seismic Risk

While often used as synonyms, it is important to distinguish between "hazards" and "risks". Hazards are natural phenomena that might impact a region, regardless of whether anyone is around to experience the outcome. Pertinent examples include earthquakes, major storms and associated wind, rain, snow, or ice, floods, and landslides. Risk refers to what we stand to lose when a natural phenomenon occurs. It is the structures we have built, their contents, the public, and the environment that are at risk from hazards.[1] A thorough risk assessment requires analysis of the hazard(s) and the anticipated performance of specific civil works when subjected to forces induced by the hazards.

The Pacific Northwest lies at the seismically-active western margin of the North American continental plate. Seismic activity is the result of tectonic activity associated with movement of earth's crustal plates. Seismic activity in the Pacific Northwest is related to three general types of earthquakes, each defined by its own style of faulting and spatial configuration relative to the plate boundaries. The three sources of primary interest in the Portland region are: Crustal Faults, Deep Intraplate faulting, and the Cascadia Megathrust earthquakes. Earthquake hazards represent the greatest risk to the Port of Portland (POP) and to the region when compared to all other natural forces or human acts. Seismic events dominate the aggregate risk exposure to civil works with impacts for life-safety, local and regional economy, and the environment.

The seismic performance of facilities at, and adjacent to, the Port of Portland (airfields, buildings, utilities, piers and wharves, levees, highway structures) will depend on the nature of the seismic loading (strength of shaking, frequency content, and duration of the motions) and the capacity of the structure(s) of interest. The latter is related to the specifics of seismic design and foundation performance. Strong ground motions can result in:

1. Structural damage, and even collapse, of buildings and other structures
2. Damage to building contents and systems that can disable operational functionality or render a building unusable
3. Ground failures due to liquefaction and lateral soil movement and settlement leading to damage to structures, slabs on grade, underground utilities, airfield pavements, roads, and embankments.

Beyond this immediate damage are the impacts of earthquakes to the community and the economy. The built environment at risk is the infrastructure that supports our economy and shelters our community. The longer it takes to rebuild the infrastructure after a natural disaster, the longer it takes the economy and community to recover. The Port's role as the gateway to goods and services for the region make it crucial to a recovery effort.

[1] http://pnsn.org/outreach/earthquakehazards

Until the early 1990s, the State of Oregon was historically considered a region of low seismicity relative to bordering states. While Washington and California have experienced sizeable earthquakes with regularity, the historic record of earthquakes in Oregon indicated relatively low rates of seismicity. This led building officials to believe that earthquake hazards were not a major concern in Oregon. As geologic investigation of prehistoric earthquakes (termed *paleoseismicity*) and other research has progressed over the last few decades, knowledge of regional faults and discoveries of evidence for large prehistoric earthquakes have improved our understanding of the regional earthquake hazard. This information has led to an evolution of the Building Code as illustrated by the increases in seismic base shear design load (Figure A). It wasn't until the early 1990s that the Oregon Building Code first required buildings to be designed for a seismic force consistent with what is now understood to be the actual hazard level.

Seismic base shear is the percentage of the weight of a structure (a force) applied as a horizontal load for purposes of designing the earthquake lateral load-resisting system for a structure to resist collapse. Figure A illustrates how the building code has increased the seismic base shear design requirement in multiple steps since the 1950s as understanding of earthquake hazards improved. With design requirements increasing over time, the capacity of older existing structures to resist even moderate earthquakes is questionable.

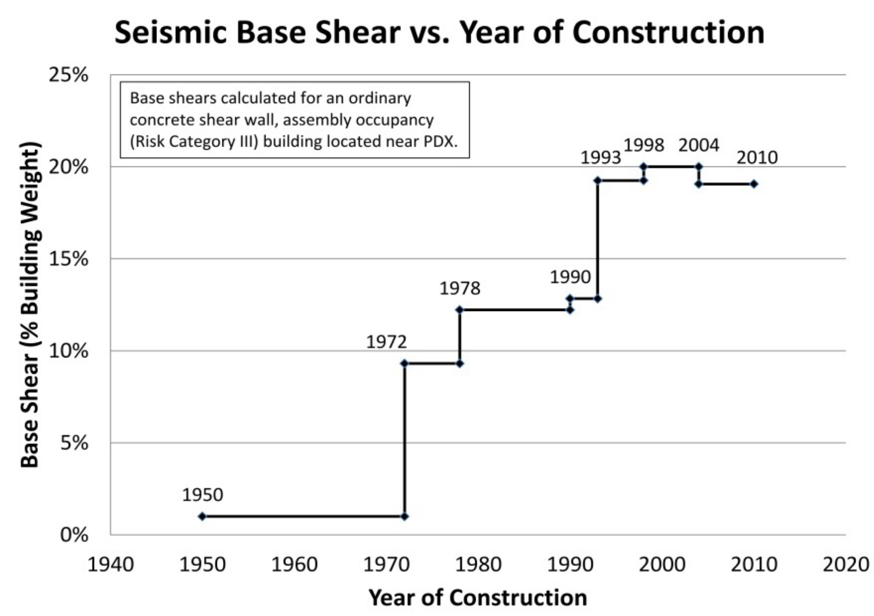

Figure A – Increase in Building Code Seismic Loading Requirements as applied in Oregon (prepared by KPFF)

Building Code: Design Performance and Structural Damage

Per current building codes (ASCE 7-10, 2012 IBC, 2014 OSSC), new buildings are designed to "perform adequately in" or "survive" a prescribed level of seismic loading without collapse. The terms "perform adequately in" or "survive" can be deceptive to non-engineers who infer some level of functionality of a structure following an earthquake, and this may not be the case after the structure has been subjected to design-level motions. Repairability and continued function of a structure are not the performance objectives of basic code seismic design. The Oregon Building Code, based on the International Building Code (IBC) serves as a minimum design standard for the design of buildings and structures in the State of Oregon. With respect to seismic base shears mentioned previously, the focus of the building code is collapse prevention under the ground motion produced by the Maximum Considered Earthquake (MCE), and life safety under the Design Earthquake (DE). Common public perception may be that a building designed to current code is “earthquake-proof”. However, the basic objective of the code is to protect against loss of human life, not to protect buildings, building contents or functions, or to ensure continued business operations. Figure B is a general illustration of building performance related to the terminology used in seismic codes and standards. Simply stated, a building that is designed and constructed to the current building code will “perform adequately” if it permits the evacuation of the occupants after experiencing design-level motions. The building may be unusable for some period of time after the design-level event. Depending on the characteristics and magnitude of the seismic ground motions and behavior of the foundation soils, a range of repairs from window replacements and drywall repairs through major structural repairs can be expected even though a structure performed adequately per the building code. In some cases, a structure that performed as designed in accordance with the code may be damaged to an extent that demolition and reconstruction are required.

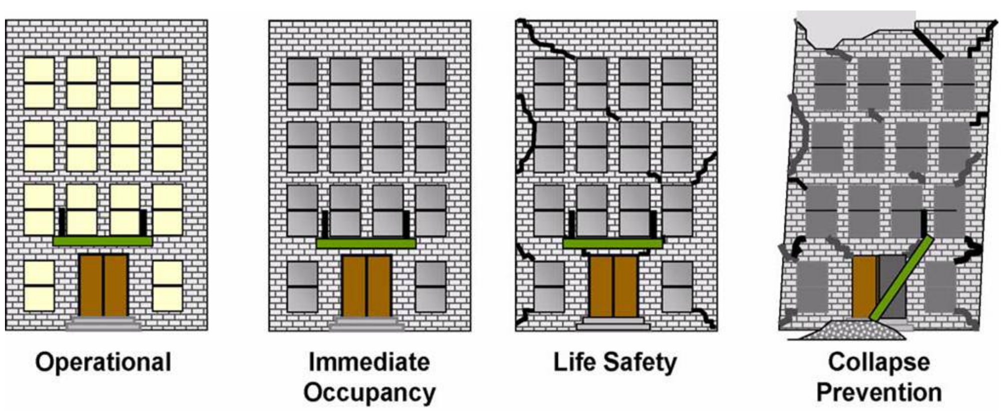

Figure B – Building Performance

Site Conditions and Potential Damage – Liquefaction and Lateral Spread

A majority of Port of Portland assets are constructed on loose, sandy soils that are vulnerable to liquefaction and loss of bearing strength. When these soils are saturated by ground water and subjected to cyclic loading, liquefaction can occur resulting in vertical subsidence, lateral spreading, and slope failures Liquefaction hazards have been identified as a primary contributor to seismic risk in waterfront areas in the Portland region.

Buildings that are supported on piles have better ability to resist the effects of earthquake-induced settlement and lateral spreading. Most critical Port buildings at PDX are pile-supported, although the capacity and depth of the piling varies widely. In general, the newer buildings have deep foundations that have greater seismic capacity than older facilities and would be expected to perform better in the absence of ground failures. Most of these buildings, however, have floor slabs that are supported directly on surface soils, and the floor slabs could experience substantial cracking from settlement of the supporting soils after earthquake loading. Older pile-supported structures may not penetrate liquefiable materials to the firmer underlying strata, and while they are much better than spread footings, they may still experience settlement and damage. Therefore, older pile-supported buildings as well as buildings on shallow spread footings at the Port face considerable risk in even a moderate earthquake. Additionally, airfield pavements, roadways, buried utilities, as well as sloped surfaces at the Port's marine facilities, are at considerable risk.

Evaluating Risk and Risk Strategies

Many significant earthquakes have occurred over the past 20 years, including several on the west coast of the U.S. Most recently, though, the earthquakes in Sumatra in 2004, China in 2008, Haiti in 2010, Chile in 2010, Christchurch New Zealand in 2010/2011, and Tohuku Japan in 2011 have heightened awareness of the potential for damage and destruction, loss of life, and economic hardship that can result from powerful seismic events. The Cascadia Subduction Zone is capable of producing such powerful seismic events and lies directly off our coast. The preponderance of evidence indicates these events have an average recurrence of between 300 and 500 years. The last event was a Mw 9+ that occurred on January 26, 1700, over 300 years ago.

The recognition that an earthquake can occur in Portland at any time has led local leaders of many government and corporate entities to assess the seismic risks to their assets. Assets can include both existing facilities and facilities that are in planning or design stages. Risk assessments consist of determining the vulnerability of an asset to earthquake damage, examining the hazards and the structural response and damage, and evaluating the life-safety and economic consequences of that damage. With the risks understood, the most appropriate or prudent strategy for mitigating risk can be identified. In light of this, the State of Oregon recently established the Oregon Seismic Safety Policy Advisory Commission (OSSPAC) to evaluate seismic risks and recommend strategies to prepare the State for a major seismic event.

Strategies for dealing with risk can range from do-nothing to extensive mitigation. Risk strategies include:

- Accepting the risk, with or without a self-administered plan for disaster response and recovery
- Accepting the risk, and expecting the Federal Emergency Management Agency (FEMA) to provide substantial post-disaster response and recovery assistance
- Accepting the risk, and purchasing earthquake insurance to help cover costs of recovery and reconstruction
- For existing facilities, undertaking mitigation projects to reduce the risk

- For facilities to be constructed in the future, designing to achieve improved seismic resilience through adoption of performance criteria above code minimums where justified
- Enacting combinations of these various strategies

As currently implemented by the Port of Portland, accepting the risk has been the most common strategy. Major earthquakes (i.e. occurrences of ground motions causing serious damage) are considered low probability events – often with several hundred years between events of large magnitude. The cost of mitigation can at first appear large when considering the low frequency of the events and the anticipated seismic performance of the asset in question. Policy leaders and decision makers typically regard the problem as unresolvable from a practical financial perspective (i.e. in the short-term, the large costs are expenses not offset by additional revenue generation). Mitigation strategies are generally difficult to integrate into business planning models. Policy and decision-making are influenced by an underlying expectation that the federal government will provide substantive response and assistance after a disaster. FEMA can be helpful, as has been seen in responses to recent natural disasters such as earthquakes, hurricanes, tornadoes, fires, and landslides. However, this strategy depends on unreliable funding from the federal government, typically results in a long recovery process, and is contrary to the FEMA recommendation of self-reliance. FEMA generally funds reconstruction over time and this takes control from the hands of local agencies. Additionally, the strategy of relying on FEMA's assistance does very little to reduce the financial risk and loss of revenue that are suffered from affected business interruptions.

The strategy of obtaining earthquake insurance also holds several disadvantages. Insurance coverage is generally limited to just a portion of repair and reconstruction costs and coverage for the identified level of risk may not be available. Insurance typically carries a substantial deductible borne by the owner. Reaching settlement and receiving payment of insurance benefits can be a protracted process, particularly in a post-disaster situation in which insurance adjustors are likely to be overburdened. The process of post-event investigations, estimating of losses, and negotiations can be expected to take considerable time. Additionally, being a reactive strategy, this does not help to avoid financial risk and loss of business revenue. Further, insurance does not improve the life-safety of vulnerable structures.

In light of these considerations and the Port's vital role in a post-event recovery effort, the Port must decide if it makes economic sense to proactively plan for seismic risk, specifically to take action to mitigate seismic vulnerabilities of Port assets. As a first significant step in making this decision, the Port has commissioned the Seismic Risk Assessment study that accompanies this preface. The Port is responsible for more than $2 billion in assets, all with regional importance to transportation, trade, and economy. These assets provide revenue to support the Port's operation, but also have a major economic impact within the region. This study was undertaken as the initial phase of a seismic assessment of Port assets, with a focus on understanding the benefits of mitigating seismic risk for a group of the most significant Port assets. For these assets, the study evaluated the seismic vulnerability of each, identified potential mitigation actions to reduce the vulnerability, and determined if these mitigation projects were viable through benefit-cost analysis. As demonstrated in the project report, seismic risk to the Port, to industry dependent on our aviation and maritime transportation system, and to the regional economy is substantial. It is envisioned that the results of this seismic risk investigation will serve as the point of departure for additional risk mitigation efforts at key assets, and assist in risk management and financial decision-making policies within the Port.

Port of Portland Engineering

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EXECUTIVE SUMMARY

The Port of Portland has conducted a seismic risk assessment of selected high-value Port assets. The seismic risk assessment was conducted to: 1) evaluate the seismic performance of the selected assets at multiple earthquake/ground-motion hazard levels, 2) identify potential improvements to selected assets that would mitigate hazards and enhance the seismic performance, and 3) estimate benefits of such improvements in comparison to cost of implementation. The study was intended to advance the understanding of the degree to which Port facilities are at risk of damage from a major earthquake and the potential economic benefit of undertaking projects to improve seismic resilience.

1. Port Assets Evaluated in the Seismic Risk Assessment Study

The seismic risk assessment considered 18 of the Port's approximately 230 assets. The 18 assets were selected on the basis of critical Port functions, high value, high revenue generation, and significance to the region in terms of economic impact. The assets represent both Aviation and Marine operations, and comprise approximately half the total value of all Port assets combined and 80% of the Port's revenue generation. The assets are listed below. The numbers indicate priority in terms of relative importance to the Port's operations; priorities were assigned at the outset of the study.

PDX Buildings

1. Central Utility Plant (CUP)
3. Concourse C – three sections
4. Terminal Core and South Lobby – four structurally-distinct components
5. ARFF Facility
6a. Port Headquarters Building and P2 Parking Structure North
6b. P2 Parking Structure South
13. Ground Maintenance Administration and Shops
14. Ground Maintenance Facility
15. Ground Maintenance Facility

PDX Airfield

2. Runway 10R-28L – South Runway
2. Runway 10L-28R – North Runway

Marine Facilities

7. Terminal 6 – Berths 604 and 605
8. Terminal 5 – Berth 503
9. Terminal 4 – Berths 410 and 411
10. Terminal 5 – Berth 501
11. Terminal 6 – Berth 601
16. Terminal 6 – Maintenance Warehouse
17. Terminal 6 – Electric Shop Building

Hillsboro Airport

12. Runway 13-31

In addition to representing a large majority of the Port's asset value and revenue production, these assets represent a significant regional economic impact. The assets account for an estimated $100 million in annual Port revenue. In 2011, these assets were estimated to account for an estimated $2 billion in regional economic impact. The regional economic impact was taken from the report The Local and Regional Economic Impacts of the Port of Portland, 2011, prepared by Martin Associates. It is expected that the contribution of the Port's assets to the regional economy has grown since that report was issued in early 2012.

2. Seismic Risk Assessments

For each of the facilities, the study conducted an assessment of vulnerability to earthquake damage. Assessments considered structural systems of the specific assets as well as site-specific soil conditions at each location. Together with the structural and soils evaluations, the study estimated the length of time each facility was likely to be out of service – or the "downtime" – following ground motions with a return period of 475 years. The facilities vary in age from 60 years to no more than a few years. The assessments considered both inertial lateral forces on structures and kinematic loading from liquefaction-induced settlement or lateral spreading. Given the varying ages of the structures and changes in building codes over the years, the capacity of the structures to resist lateral loads varies considerably both for PDX buildings and marine structures. Newer structures typically have the capacity to resist larger forces than older structures, as would be expected.

The entire PDX site has subsurface conditions susceptible to soil liquefaction and seismically-induced settlement. Many of the buildings at PDX have pile foundations. Typically, buildings with shorter pile foundations that do not penetrate to dense, non-liquefiable soil deposits are more vulnerable to settlement-caused earthquake damage than buildings with long pile foundations. Most of the older structures have shorter pile foundations. The majority of buildings at PDX, even those with long pile foundations, have slab-on-grade ground floors which will settle as the result of soil liquefaction and settlement. Consequently, earthquake-induced settlement of ground floor slabs can occur even in a building that is otherwise undamaged by forces of a particular earthquake.

All of the Port's marine structures are also located in areas where the soil is susceptible to liquefaction as well as to lateral spreading. The large estimated soil displacements caused by lateral spreading can impose significant, damaging forces on structural elements. In larger earthquake events, the majority of the marine facilities will likely be damaged beyond repair.

Findings of the preliminary assessments for each facility are summarized in the following:

PDX Building Assessments

Central Utility Plant: Originally constructed in 1970; expanded in 1992 and upgraded in late 1990s. Design capacity of the lateral force-resisting system for earthquake forces ranges from

65% to 87% of current code lateral design forces. The building is composed of a variety of different structural systems which could result in an undesirable distribution of lateral earthquake forces. The building lacks ductile detailing, and the thin, brittle exterior masonry walls are susceptible to damage. Pile foundations are relatively shallow, and the building may to settle several inches even in a moderate earthquake. Downtime to rebuild and repair the CUP following seismic forces from ground motions having a 475-year return period is estimated to be approximately 12 months. Additional time could be needed to procure, install, and commission specialized equipment.

An 80-foot length of corrugated steel pipe (CSP) utility tunnel exists between the CUP and the utility tunnel under the P2 parking structure (P2). The CSP utility tunnel is not pile-supported, and it will settle relative to the CUP and the pile-supported utility tunnel under P2. The differential settlement can be expected to damage utilities inside the tunnel.

Concourse C: Constructed in late 1990s. The lateral force-resisting system is steel moment-resisting frames. Design capacity is 103% of current design requirements; however, lateral drifts of the building in a relatively large earthquake will exceed current standards for Immediate Occupancy, and the movement of the building could damage glazing and other non-structural components necessary to meet Immediate Occupancy conditions. The building is supported on deep piles which will prevent significant settlement of the structure. However, the slab-on-grade ground floor will settle in the event of earthquake-induced liquefaction. The settlement will damage architectural and MEP elements that are supported by the slab. Additionally, the utility tunnel below Concourse C is not pile-supported, and is likely to settle. Downtime to restore Concourse C to an occupiable condition after the 475-year hazard level ground motions is estimated to be two months.

Terminal Ticket Lobby: Originally constructed in 1973; seismically upgraded in the mid-1990s. The lateral force-resisting system is composed of concrete shear walls with steel braced frames above the Mezzanine. Design capacity for the shear walls is 97% of current code, and for the braced frames is 63% of current code. The building lacks ductile detailing which will likely result in localized damage in a major earthquake. The original pile foundation is relatively shallow, and was supplemented with deeper micropiles in the upgrade. The shallow piles will settle when soil liquefaction occurs, causing stresses in the building structure and increased loads on the micropiles. The slab-on-grade ground floor and exit vestibules will settle, possibly by 12 inches or more in a large earthquake. Downtime to restore the building to an occupiable condition after the 475-year event is estimated to be 12 months.

Terminal South Node: Constructed in late 1990s. Design capacity is 103% of current code. However, detailing of the shear wall reinforcing may not meet current code. A lack of ductile detailing could lead to localized damage in a large earthquake. The pile foundations are deep and are not expected to settle significantly. As elsewhere in the terminal, however, the slab-on-grade ground floor will settle 12 inches or more in a large earthquake. Downtime to repair damage from the 475-year event is estimated to be two months.

Terminal Oregon Marketplace South: Originally constructed in 1956; expanded and upgraded in 1986 and 2002. The lateral force-resisting system is a combination of concrete shear walls and steel braced frames. Design capacity for the concrete shear walls is 107% of current code, and for the braced frames is 70% of current code. Similar to the Ticket Lobby, a lack of ductile detailing will result in localized damage. Piles from the original construction are shallow, and will likely settle several inches in a soil liquefaction event. The settlement will cause stresses in the building and increased loads on micropiles that were installed in the 2002 upgrade. Soil liquefaction will cause slab-on-grade settlements of 12 inches or more. Downtime to restore the building to an occupiable condition after the 475-year event is estimated to be 24 months.

Terminal Oregon Marketplace Central: Originally constructed in 1956; upgraded in 1986 and late 1990s. The ongoing phased voluntary seismic upgrade of the Terminal has not been completed in this area. If completed, the design capacities based on the existing concrete shear walls and braced frames would be 107% and 70% of current code respectively. Due to a lack of ductility in the structure, the building is expected to perform poorly in a large earthquake. The existing pile foundation is shallow, and will not prevent settlement of the building in a large earthquake. The settlement will cause extensive damage to the older concrete structure. Settlement of the slab-on-grade could be as much as 10 inches in a 500-year event. Downtime to restore the building to an occupiable condition after the 475-year event is estimated to be 24 months.

PDX Aircraft Rescue and Firefighting Facility: Constructed in the 1990s. The ARFF facility building was designed as an Essential Facility with concrete masonry shear walls. The design capacity for seismic forces is 107% of current code requirements. The structure likely does not meet current requirements for ductility, and localized damage can be expected. With a high potential for liquefaction at the site, the building can be expected to settle due to its mat foundation rather than deep piles. Settlements of approximately 6 inches could occur with ground motions having a 200-year return period, and more than a foot with larger ground motions. The building may not be significantly damaged by the settlement, but certain elements such as the doors of the truck bays may not be workable. Downtime to restore the ARFF facility following the 475-year event is estimated to be two months.

Port Headquarters (HQ) and P2 Parking Structure: Constructed in 2009. The lateral force-resisting system is a combination of concrete shear walls and concrete and steel moment-resisting frames. Design capacity is 100% of current code requirements. The building meets code design and detailing requirements and code performance expectations. Deep pile foundations will prevent settlement of the building. However, the slab-on-grade ground floor – part of the P2 Garage – may settle as much as a foot in a large earthquake. Downtime to restore HQ/P2 to an occupiable condition after the 475-year event is estimated to be one month. The estimated one-month downtime would not include repairing the ground floor.

PDX Ground Maintenance Facilities: Constructed in the 1980s. The three buildings are of generally similar construction consisting of precast, tilt-up concrete walls with plywood diaphragm roofs. Design capacity for lateral seismic forces ranges from 31% to 37% of current code design forces. Lateral systems lack ductility, and the roof structures do not meet current design standards. The site is highly susceptible to soil liquefaction. Ground settlements of as much as 18 inches could occur in an earthquake with ground motions having a return period of as little as 200 years. Spread footings could settle an additional foot. The extreme settlements together with the seismic deficiencies of the structures will likely result in the buildings being unusable after a 200-year event. Downtime to replace the buildings is estimated to be approximately 16 months.

PDX Runway Assessment

Runways 10R-28L (South)/10L-28R (North): The South Runway was reconstructed in 2011; the North Runway was extended and rehabilitated in 2009 and 2010. As noted in the PDX building assessments, the soils at PDX are highly susceptible to seismically-induced liquefaction. The resulting settlement will affect airfield pavements. Minimal damage is likely to occur when subjected to ground motions having an average return period of72 years, while differential settlements are likely to become operationally unacceptable at ground motion levels greater than approximately 225-year exposure intervals. Soil conditions are generally similar at the two runway sites, with the exception that a higher risk of lateral spreading exists at the north runway location. Repair times to return a runway to service will of course depend on the extent of damage, and could range from a few days for minor repairs to 10 -12 months for full reconstruction. Repairs to the asphalt concrete North Runway will likely require less time in general than repairs to the portland cement concrete South Runway.

Marine Facility Assessments

Marine structures, with the exception of Terminal 6 – Berths 604/605, were assessed in this study for performance at 72-year, 475-year, and 975-year return period ground motions in accordance with current industry approach. Berths 604/605 were evaluated in an earlier study conducted by the Port. In general, all of the facilities in their existing condition will experience some degree of damage from a 72-year hazard level ground motions, and none would be expected to survive a 475-year event.

Terminal 6 – Berths 604/605: Originally constructed in 1974; modified in 1994 and 2011/2012. Berths 604 and 605 are sand-filled cellular sheet pile structures. Ground improvements to increase seismic resilience of an 800-foot section of the wharf were undertaken by the Port in 2011 and 2012. Based on evaluations conducted previously by the Port, the improved section should survive ground motions at a 200-year return period. The unimproved section of the facility is expected to be vulnerable to damage beyond a 50-year event.

Terminal 5 – Berth 503: Constructed in 1992. Original design criteria for Berth 503 are unknown. It can be expected that the criteria were considerably below current code requirements. The structure is composed of concrete piles, concrete pile caps and beams, and a concrete deck with isolated steel batter pile elements. The structure is expected to survive 72-

year ground motions with relatively minor damage. Downtime for repairs following the 72-year event is estimated to be 5 to 8 months. A recent evaluation conducted for the facility indicates that ground motions at the 475-year return period, without consideration of the effects of liquefied soils, would cause forces in the structure at or slightly above capacity. At this return period, soil liquefaction will result in lateral spreading estimated to be on the order of seven feet. This extent of soil displacement may cause substantial damage, such that the berth may not be repairable. A 26- to 38-month downtime for replacement can be expected.

Terminal 4 – Berths 410/411: Berth 411 constructed in 1959; Berth 410 constructed in 1962. Design capacity for the lateral systems is approximately 30% of current code design forces. Structural systems vary, with Berth 410 constructed primarily of timber elements and Berth 411 constructed of concrete elements. However, the performance of the two berths is expected to be similar. The structures will likely survive a 72-year return period event with repairable damage. The 475-year event will induce significant soil liquefaction which will cause large lateral soil displacements. The soil displacements will result in excessive forces on structural elements. The facilities are not expected to survive the 475-year event. Downtime to reconstruct the berths is estimated at 26 to 38 months.

Terminal 5 – Berth 501: Constructed in 1974. Design criteria for this facility are unknown, but are likely to have been well below current code. The facility is a hybrid pier structure consisting of three large-diameter sheet pile cells supporting a concrete deck. Earthquake-caused liquefaction at the site will induce large lateral soil deformations resulting in significant forces on the sheet pile cells. The 72-year event will likely cause significant damage requiring extensive repairs. Downtime to repair damage from the 72-year event is estimated to be 12 to 16 months. The 475-year event will likely damage the pier beyond repair. Reconstruction time is estimated to be 22 to 34 months.

Terminal 6 – Berth 601: Constructed in 1989. Berth 601 is a floating dock with a trestle connection to the shore. The floating dock will not experience significant damage from an earthquake, as a result of being waterborne. Design capacity for the lateral system of the trestle is approximately 11% of self-weight, which is approximately equal to current code forces for the 72-year return period event. Soil lateral spreading displacements at the site will be extensive, estimated at several feet from ground motions at the 72-year event and in excess of 10 feet at the 475-year event. The trestle and other landward elements are expected to suffer significant damage from the soil displacements in the 72-year event, and may not survive. Downtime to construct a new trestle and replace other landward elements is estimated at 15 to 21 months.

Terminal 6 Maintenance Warehouse: Constructed in the 1970s. Design capacity, originally based on wind loading, ranges from 35% to 77% of the current code seismic design forces. The lateral system is composed of a combination of tension rod bracing and steel moment frames. The design lacks the ductile configuration and detailing required by current code. Additionally, soil liquefaction could lead to settlements exceeding a foot in ground motions at a return-period of less than 300 years, resulting in significant damage. The building is not likely to survive beyond a 200-year event. Downtime to replace the building is estimated to be 12 months.

Terminal 6 Electrical Shop: Constructed in the late 1980s. The building was designed for wind loading, similar to the Maintenance Warehouse. Design capacity is 167% of the current code seismic design forces in one direction, but only 28% of the current code in the other direction. The lateral system consists of tension rod bracing and moment frames, which lack the ductility required by current code. As noted for the Maintenance Warehouse, large settlements at the site will likely occur in relatively small earthquakes. The building is likely to be damaged beyond repair in a 200-year event. A 12-month replacement downtime would be expected.

Hillsboro Airport Runway Assessment

Hillsboro Runway 13-31: Soils at the site of the Hillsboro Airport are less prone to seismically-induced liquefaction and settlement than the soils at PDX. Screening-level analyses indicate that there is a low risk of significant soil settlement at the Hillsboro site. A magnitude 9.0 earthquake is likely to cause some runway settlement but not take the runway out of service. Portions of the runway may need to be repaired to return the runway to original condition, but such repairs will not likely need to be undertaken immediately to maintain the runway in service.

3. Seismic Risk Mitigation Strategies for Selected Assets

The study identified potential strategies to mitigate the expected seismic risk for a selected group of the assets evaluated. The selected group of assets included the CUP, Concourse C, sections of the main passenger terminal, and the South Runway at PDX, and marine Terminal 4 – Berths 410/411, Terminal 5 – Berths 501 and 503, and Terminal 6 – Berth 601. For buildings, seismic risk mitigation was targeted at achieving a condition of Immediate Occupancy for ground motions having a 475-year return period. For marine facilities, the objective of mitigation was to achieve survivability for the 475-year return period event.

Seismic risk mitigation for the Port's assets will generally entail both improvements of structural systems and improvements of soils. At all PDX and marine facilities, mitigation must necessarily address the liquefaction potential of the soils. The soils are deep alluvial flood deposits of the Columbia River and Willamette River, and as noted in the foregoing narrative are highly susceptible to liquefaction. The ground settlements and lateral spreading that are triggered by liquefaction can be damaging to all types of structures.

The potential mitigation strategies identified in the study are summarized in the following:

Central Utility Plant: Improve the foundation to prevent settlement of the building by installing deep micropiles at each column and other load-bearing elements, and at locations of critical equipment. Strengthen the lateral capacity of the building by retrofitting with a concrete shear wall system. Replace the brittle exterior wall system composed of masonry blocks and brick veneer with a more flexible system such as metal studs and metal panels. Improve anchorages and support for essential MEP equipment and systems. A rough order of magnitude estimate of probable construction cost for these actions is $16 million. The estimate is based on 2014/2015 costs.

In addition to the CUP, risks to the unsupported utility tunnel that exists between the CUP and the parking structure should be addressed. A new pile-supported concrete tunnel could be constructed around the existing tunnel to eliminate settlement potential.

<u>Concourse C</u>: Install micropiles under the slab-on-grade ground floor to prevent significant settlement. Alternatively, reinforcing the slab with a reinforced topping slab bonded to the existing slab would be feasible in some areas. Install micropiles to support the utility tunnel. For the lateral system of the building, install a force damping system to improve seismic performance. Some additional bracing of critical MEP systems would be needed. Order of magnitude estimate of cost: $81 million total for all three sections of the concourse.

<u>Terminal Ticket Lobby</u>: Install micropiles at each column and other load-bearing element to prevent settlement of the building. Install micropiles under the slab-on-grade ground floor to prevent settlement, or replace the slab with a structural slab. Replace steel braced frames in the lateral system with more ductile braces for better ductility performance, and reinforce certain structural connections. Additional bracing of critical MEP systems would be needed. Order of magnitude estimate of cost: $47 million.

<u>Terminal South Node</u>: Install micropiles under the slab-on-grade to prevent settlement, or replace the slab with a structural slab. Install micropiles to support the utility tunnel. Improve structural diaphragm connections to improve the strength and ductility of the lateral structural system. Additional bracing of critical MEP systems would be needed. Order of magnitude estimate of cost: $36 million.

<u>Terminal Oregon Marketplace South</u>: As for the Terminal Ticket Lobby, install micropiles at each column and other load bearing elements to prevent settlement. Install micropiles under the slab-on-grade. For the lateral system, replace braced frames with more ductile bracing, with the exception of two braced frames that should be replaced with concrete shear walls. Additional bracing of critical MEP systems would be needed. Order of magnitude estimate of cost: $20 million.

<u>PDX Runway</u>: Install stone columns or jet grout the supporting soil. At either the North Runway or the South Runway, stone columns would extend to a depth of approximately 40 feet below the pavement surface. Jet grouting treatment would extend to a depth of approximately 30 feet. Stone columns would be installed as part of a scheduled reconstruction project; jet grouting could be undertaken as a retrofit. Order of magnitude estimates of cost: $137 million for jet grout treatment of the South Runway, $67 million for stone column improvements for the South Runway, and $68 million for stone column improvements for the North Runway.

<u>Terminal 4 – Berths 410/411</u>: Given the age of these facilities and the cost of improvements that would be needed to achieve survivability at the 475-year return period, the only mitigation

action that would be economically viable is to replace the berths with a modern facility. It is expected that replacing the two berths with a single combined facility would be the preferred approach. Order of magnitude estimate of cost for a combined replacement facility: $42 million.

Terminal 5 – Berth 501: Conduct ground improvements to limit soil displacements. Ground improvements could consist of installing stone columns or other strengthening method in the river embankment, around the trestle abutment, and in the cellular structures. Install new piles to support the conveyor bridge tower, and strengthen structural members and connections throughout the facility. Order of magnitude estimate of cost: $20 million.

Terminal 5 – Berth 503: Conduct ground improvements along the shoreline using stone columns, and strengthen piles, pile connections, and concrete beams. Order of magnitude estimate of cost: $13 million.

Terminal 6 – Berth 601: Conduct ground improvements using stone columns around the approach trestle bents and abutments, and install piles at each bent. Retrofit the trestle structure by strengthening structural elements and connections. Order of magnitude estimate of cost to retrofit the trestle with new piles and stronger connections: $5 million.

It should be noted that mitigation strategies other than those mentioned here were considered and may be appropriate; discussion of other strategies was omitted in the interest of brevity. Future work and additional in-depth studies by the Port would determine the optimal mitigation strategy for any asset.

4. Risk and Benefit-Cost Analyses

The study conducted risk and cost-benefit analyses of the assets and the potential mitigation strategies. The analyses were conducted to evaluate the benefits of mitigation by comparing existing "as is" conditions with the mitigated conditions. Eight different cases were evaluated, considering Port-only revenue impacts and Port-plus-Region combined economic impacts:

a. Port Only Impacts – Buildings, Existing ("As-is")
b. Port Only Impacts – Buildings, Runways and Marine Facilities, Existing ("As-is")
c. Port Only Impacts – Buildings with Mitigation
d. Port Only Impacts – Buildings, Runways and Marine Facilities with Mitigation
e. Port and Regional Impacts – Buildings, Existing ("As-is")
f. Port and Regional Impacts – Buildings, Runways and Marine Facilities, Existing ("As-is")
g. Port and Regional Impacts – Buildings with Mitigation
h. Port and Regional Impacts – Buildings, Runways and Marine Facilities with Mitigation

Benefit-Cost for Mitigation of PDX Assets

The total order of magnitude estimated cost of the potential mitigation strategies identified in the study for the PDX assets is $267 million. The assets include the CUP, Concourse C, the three units of the passenger terminal, and the South Runway.

	Estimated Cost of Mitigation
CUP	$16,000,000
Concourse C	$81,000,000
Terminal Ticket Lobby	$47,000,000
Terminal South Node	$36,000,000
Terminal Oregon Marketplace South	$20,000,000
South Runway	$67,000,000
Total	$267,000,000

Considering Port plus regional economic impacts, benefit-cost analysis shows a benefit-cost ratio of 1.4 for the combined mitigations. A benefit-cost ratio of 1.4 represents a relatively good payback on investment, on the basis that a ratio greater than 1 indicates a positive economic benefit.

A comparison of the cost-effectiveness of the potential mitigation actions for each of the PDX building assets showed that the greatest benefits in loss reduction would be produced by mitigations for the CUP, the Terminal Ticket Lobby, and the Terminal Oregon Marketplace South. The order of magnitude cost estimate for mitigations of these three building assets is $83 million. With the South Runway mitigation at $67 million, the total cost of the mitigation strategies for this smaller group of assets would be $150 million. Considering Port and regional economic impacts, the benefit-cost analysis shows a benefit-cost ratio of 2.2 for risk mitigation for this smaller group.

Benefit-Cost for Mitigation of Marine Assets

Benefit-cost analysis for the potential retrofit mitigation actions at all of the marine facilities evaluated, with the exception of Terminal 4 – Berths 410/411, shows benefit-cost ratios greater than 1 considering Port and regional economic impacts. Specific benefit-cost ratios for mitigation actions are as follows:

	Estimated Cost of Mitigation	Benefit-Cost Ratio
Terminal 4 – Berths 410/411	$42,000,000	0.8
Terminal 5 – Berth 501	$20,000,000	3.5
Terminal 5 – Berth 503	$13,000,000	1.8
Terminal 6 – Berth 601	$5,000,000	2.9
Terminal 6 – Berths 604/605	$15,000,000	2.2

As noted, the only economically viable mitigation strategy for Terminal 4 – Berths 410/411 is complete replacement. The cost of facility replacement and the time out of service take the benefit-cost ratio for that action below 1.

5. Conclusions and Recommendations

The Seismic Risk Assessment Study identified risks of seismic damage in the majority of the Port assets evaluated. Given the importance of the Port's function to the region, it is recommended that the Port continue with actions to improve the seismic resilience of key Port assets. For PDX, the focus could be on improving the resilience of a group of assets that would together represent a functional airport – a portion of the passenger terminal, a concourse, the CUP, and a runway. The Terminal Ticket Lobby, Terminal South Node, Terminal Oregon Marketplace South, Concourse C, the CUP, and either the North Runway or the South Runway would fit this description. For marine facilities, the focus could be on protecting the assets that provide the greatest revenue and functionality. Accordingly, it is recommended that the Port give consideration to the following specific mitigation projects:

PDX Runway

Mitigation of risks to a PDX runway should be a top priority. Given the liquefaction potential of the ground at PDX, relatively low to moderately strong ground motions will cause ground settlement and distortion of pavement to some extent. This would result in a high probability of a repair project that would take the runways out of service for some length of time. Without a usable runway, the airport would not be functional. Further study would determine if the mitigation should be for the South Runway or the North Runway. Planning for a runway mitigation project should include discussions with the FAA about physical condition requirements for a runway to remain in service after an earthquake, and about the potential for improving the survivability of critical FAA-owned navigational aids.

PDX Terminal

A terminal mitigation project should be pursued as a second priority. The terminal is necessary for passenger check-in functions and baggage handling. The focus of a mitigation effort should be on terminal units T1 – Ticket Lobby and T3 – Oregon Marketplace South, for which mitigation actions show the greatest cost-effectiveness. The mitigation could be part of the Terminal Core Redevelopment project that the Port has initiated; that project would provide an avenue and mechanism to accomplish the seismic retrofits.

PDX Central Utility Plant or Concourse C

A mitigation project for either the CUP or Concourse C should be a third priority. A functioning CUP is critical for full operation of the terminal and airfield functions. Further study would confirm the vulnerability of the CUP and determine the optimal retrofits. If the further study finds that the CUP is not as vulnerable as believed, consideration should be given to mitigating the risks at Concourse C as the third priority.

Marine Terminal T6 – Berths 604/605

Mitigation at Terminal 6 – Berths 604/605 should be completed, as the top priority for the Port's marine assets. A portion of the wharf has been seismically upgraded. A project to mitigate risks for the remainder of the wharf would improve the resilience of the entire facility to withstand a large earthquake. Berths 604/605 would likely be the most important Port marine asset in supporting a regional rebuilding effort in the aftermath of a major disaster.

Marine Terminal T5 – Berth 503

Mitigation at Terminal 5 – Berth 503 should be the second priority for the marine assets. Berth 503 operates under the most stable, long-term lease of the Port's marine facilities. Seismic vulnerabilities should be mitigated to keep this facility in business for the long term.

For each of these potential projects, next steps will include detailed geotechnical site assessments, detailed structural engineering analyses, and further explorations of potential mitigation measures, costs, and benefits.

Beyond the specific, prioritized project recommendations outlined in the foregoing, this study offers the following additional recommendations:

- Evaluate the benefit of designing each new project for greater seismic resilience than the minimum required by Building Code. Considering that code requirements for seismic design forces are based on life-safety and collapse prevention, not on property preservation or operational continuity, structures designed to minimum code requirements cannot be expected to maintain uninterrupted functionality after a major earthquake.
- Identify and evaluate mitigations for other key Port assets. This study identified and evaluated potential mitigation actions for only a limited number of the Port's key assets. A similar effort should be undertaken for other assets considered to be critical for the Port's functions.
- Establish a plan for extricating aircraft rescue and firefighting vehicles from the ARFF facility if the doors of the truck bays become inoperable after an earthquake.
- Broaden future seismic risk assessment efforts to include non-Port critical assets and lifelines, in coordination with other agencies and with utility owners. Pertinent examples include:
    - Services provided by regional lifeline systems such as electrical power, telecommunications, water/wastewater, fuel, and surface transportation
    - The Columbia River levee system adjacent to PDX
    - Jetties at the mouth of the Columbia River, and navigation channels along the full lengths of the Columbia River and Willamette River shipping lanes
- Confirm the plan for Port emergency operations and recovery. Immediate occupancy after a significant ground motion should not be expected for any Port facility as it currently exists. The Port should assess the current emergency response plan to ensure there is an allowance for the probable temporary unavailability of existing Port facilities.

End of Executive Summary

1. INTRODUCTION

The Port of Portland has conducted a seismic risk assessment study of selected key Port assets. This report documents the study, presents the findings, and sets forth conclusions including recommendations for potential risk mitigation projects for consideration.

1.1 Study Objectives

The overall objectives of the seismic risk assessment were to understand the general vulnerability to major earthquakes of selected critical Port facilities and to accomplish benefit-cost analyses of potential seismic risk mitigation actions for a portion of the selected facilities. Business continuity and lifeline planning necessitate that the Port understand the seismic fragility of the Port's assets, particularly vital high revenue and critical lifeline assets, and know the costs of business interruptions, the actions that could be taken to mitigate the risks, and the benefit-cost ratios of such mitigating actions. To that end, the seismic risk assessment:

- Identified a list of key Port assets to consider, representing both Aviation and Marine operations
- Assessed the seismic fragility of the identified key Port assets, considering several seismic hazard levels
- Estimated duration of service loss or downtime
- Estimated costs of repair or replacement
- Estimated economic losses resulting from business interruptions, considering economic losses both to the Port and to the region
- Identified potential mitigation actions for selected assets and developed order of magnitude estimates of cost for the actions
- Conducted benefit-cost analyses of potential mitigation actions
- Produced benefit-cost ratios for the potential mitigation actions
- Identified specific mitigation projects for further study and analysis potentially leading to incorporation into the Port's capital improvement program

The Port has previously conducted limited seismic risk evaluations of Port facilities, most notably a detailed analysis of particular elements of Terminal 6 and a code assessment of Port-owned buildings at PDX. With the goal of balancing project resources with an "advanced screening level" of seismic performance assessment, this study was undertaken to a level of analysis that was between the detailed Terminal 6 analysis and the more general assessment of PDX buildings. This level of seismic performance assessment was deemed appropriate for accomplishing the engineering-economic analysis, and for identifying assets that may warrant additional, more refined analysis of seismic performance. The study focused on selected key assets as gauged by importance to business continuity, revenue production, and lifeline support. The Port assembled a Technical Advisory Committee to guide the study. The Advisory Committee was composed of Port asset managers, risk managers, and emergency response managers. The Advisory Committee participated in finalizing the scope of the study, identifying risk parameters, prioritizing assets, defining seismic scenarios for scenario-based analysis, and reviewing results of the study through three focused workshops conducted over the course of the study.

In addition to developing a general understanding of the seismic vulnerability of the selected priority assets and evaluating potential risk mitigation projects, it is expected that this study can serve as a framework for seismic risk assessments of other Port assets the Port may conduct in the future. This study is unique, not only to the Port but to public agencies in general. The study began with an outline approach that was refined and adjusted by the project team in consultation with the Port's Technical Advisory Committee over the course of the work. Future studies may follow the approach of this study, no doubt making improvements with gained experience and knowledge.

Additionally, the study may draw attention to seismic performance expectations in comparison to building code requirements, particularly in relation to new construction. Building codes establish a minimum seismic force for design of new structures. The minimum seismic design force is based on life-safety and collapse-prevention considerations. New structures designed to code cannot be expected to necessarily remain operational following a design-level earthquake, or even be in condition for immediate occupancy. To achieve Immediate Occupancy, not only must the structure survive with little damage, but necessary utilities must be functioning and other non-structural components such as glazing, ceilings, and partitions must remain in serviceable condition. An understanding of this introduces the need to evaluate, for each new project, the benefits of designing the project to levels of seismic resiliency above the minimums required by code.

1.2 Assets Considered in the Study

The seismic risk assessment focused on a select group of the Port's assets. The assets to be evaluated were selected on the basis of critical Port functions, high value, high revenue production, high regional economic impact, and importance to the region in the aftermath of a major earthquake. The Port's facilities will serve three critical roles following a catastrophic seismic event: 1) accommodating the inflow of initial recovery response and relief supplies; 2) facilitating resumption of Port business activities and minimizing business interruption impacts; and 3) serving as the gateway for the goods and services that would be required to rebuild the community and restart the regional economy. The Port's 230-plus assets were previously evaluated and prioritized with respect to Port functions, value, revenue production, and regional importance. This study focused on the top 18 assets in terms of priority. The 18 assets, representing both Aviation and Marine operations, are summarized below.

1.2.1 Portland International Airport Facilities

PDX is the Port's primary revenue generator, and would be expected to serve as a key hub for recovery efforts in the wake of a major earthquake. The seismic risk assessment study identified a group of priority Port assets that represent a large portion of the value of PDX, and that also would together form a functional subset of PDX assets that would enable operations to be conducted following a major earthquake if the assets maintained functionality. The subset of assets is composed of Concourse C for aircraft gates, a portion of the main terminal as outlined below to process passengers, the ARFF station, the Central Utility Plant, Ground Maintenance facilities, and a runway. Concourse C was selected because it is the most recently constructed of the PDX concourses, provides the most aircraft gates, and shelters Port police and emergency operations. Both Runway 10R-28L (South) and Runway 10L-28R (North) were considered in the assessments. For the subset of assets selected to comprise a working airport, the South Runway was selected because it is the longest runway at PDX, is the newest runway, and has the greatest instrument landing system capability.

An assumption in selecting the assets was that the Port's goal would be to maintain operations following an earthquake in a fashion similar to existing conditions, such as serving mainline passenger aircraft with passenger boarding bridges. It is understood that airport operations could occur with a lesser compliment of facilities; for example, passenger aircraft could be served by stairs from the ground rather than by boarding bridges from a concourse building. However, this would represent a considerable reduction in functionality and capacity, with a commensurate reduction in service level and revenue. Accordingly, the study focused on facilities to maintain the current form of functionality and service.

The PDX assets included the following, with priority rankings as indicated:

<u>PDX Buildings</u>

1. Central Utility Plant
3. Concourse C – comprised of three sections separated by seismic isolation joints
   - Concourse C East
   - Concourse C Central
   - Concourse C West
4. Passenger Terminal Building – consisting of selected sections of the building (separated by seismic isolation joints) that would comprise a usable terminal, including:
   - Terminal Ticket Lobby
   - Terminal South Node
   - Terminal Oregon Marketplace South
5. Aircraft Rescue and Fire Fighting (ARFF) Facility
6. Port Headquarters Building and Second Parking Garage (P2)
13. 6a. HQ and P2 North 6b. P2 South Ground Maintenance Administration and Shops
14. Ground Maintenance Facility
15. Ground Maintenance Facility

<u>PDX Airfield</u>

2. PDX Runway
   - Runway 10R-28L – South Runway, or
   - Runway 10L-28R – North Runway

1.2.2 Marine Terminal Facilities

Marine assets addressed in the study are listed below. Information regarding seismic assessment of Berths 604 and 605 at Terminal 6 was taken from an earlier study conducted by the Port.

7. Terminal 6 – Berths 604 and 605
8. Terminal 5 – Berth 503
9. Terminal 4 – Berths 410 and 411
10. Terminal 5 – Berth 501
11. Terminal 6 – Berth 601
16. Terminal 6 – Maintenance Warehouse
17. Terminal 6 – Electric Shop Building

1.2.3 Hillsboro Airport

Hillsboro Airport is not a major revenue generating asset for the Port. However, due to the location of the airport west of the Willamette River and potentially damaged bridges, the airport was considered as a priority site to support or assist with emergency response and recovery following a major seismic event. The study identified the primary runway at Hillsboro Airport as a priority asset to be considered.

12. Runway 13-31

The general locations of the PDX and marine assets are depicted on Figure 1.

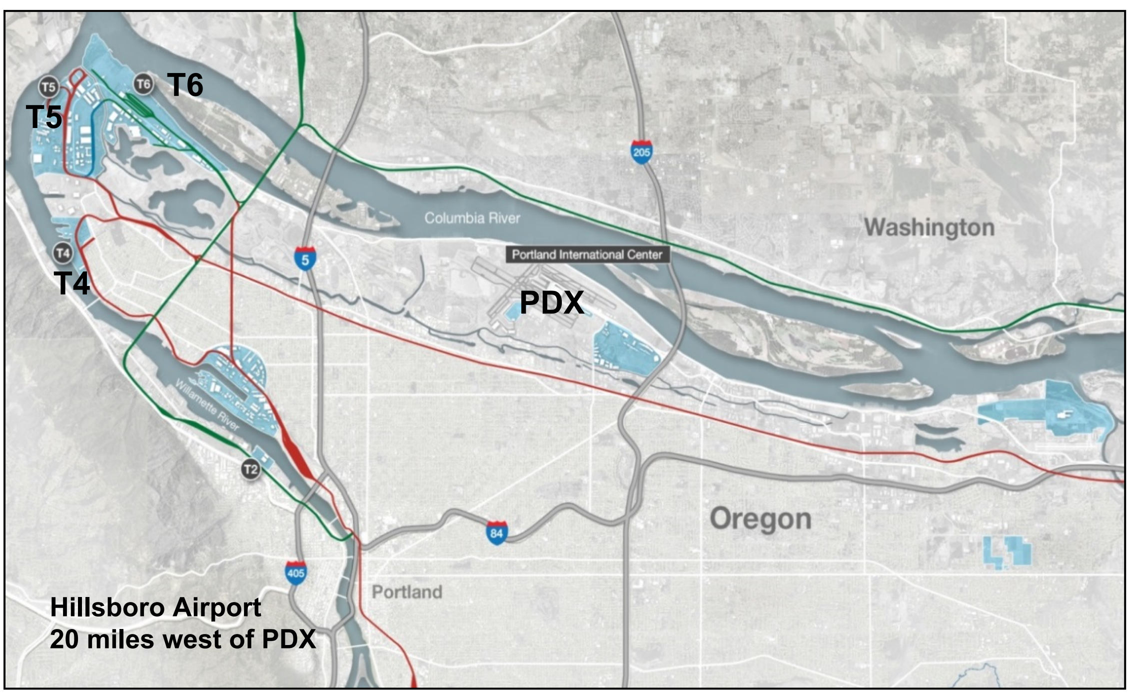

Figure 1 – General Location of PDX and Marine Assets

1.3 Asset Financial Data

The 18 assets represent approximately one-half the value of the Port's total of approximately 230 assets, and account for approximately 80% of Port revenue. Table 1 lists financial data for the assets that were used in the benefit-cost analyses. The minimum replacement values shown in the table were provided by the Port Risk Department, and represent insurance coverage. The engineering estimated total replacement cost is the estimated project cost to completely replace an asset with a new asset for the same function. The replacement estimates represent order of magnitude costs at 2014/2015 values. Estimated annual Port revenue figures listed in the table were obtained from terminal managers, business line managers, and Port finance personnel. Annual regional economic impact figures were obtained from the report entitled The Local and Regional Economic Impacts of the Port of Portland, 2011[2].

Table 1 – Asset Costs, Revenue, and Economic Impact

Facility	Asset	Priority Rank	Minimum Replacement Value (Insurance Coverage)	Engineering Estimated Total Replacement Cost	Estimated Annual Port Revenue	Estimated Annual Regional Economic Impact
PDX	Central Utility Plant Bldg & Mech Tunnel	1	$38,000,000	$63,000,000	$80,800,000	$1,560,000,000
	Airfield Runways, Taxiways, Ramps & Lighting	2	$280,000,000	$110,000,000		
	Terminal Concourse C and Pass Structure	3	$160,000,000	$543,000,000		
	Main Passenger Terminal Building	4	$140,000,000	$936,700,000		
	ARFF (Fire) Station	5	$13,000,000	$15,200,000		
	P2 Parking Structure	6b	$127,000,000	$196,000,000		
	Ground Maintenance Admin & Shops	13	$6,300,000	$11,700,000		
	Ground Maintenance Facility	14	$2,800,000	$5,580,000		
	Ground Maintenance Facility	15	$200,000	$2,850,000		
T6	ICTSI - Berths 604 and 605	7	$23,000,000	$100,000,000	$11,000,000	$120,000,000
	Yard trailer maintenance	16	$1,500,000	$2,970,000		
	Electric Shop Bldg, SW of Admin Bldg	17	$4,000,000	$1,905,000		
	AWC - Berth 601	11	$8,000,000	$35,000,000	$2,000,000	$18,000,000
T5	Portland Bulk Terminal - Berth 503	8	$20,000,000	$20,000,000	$2,500,000	$98,000,000
	Columbia Grain Facility - Berth 501	10	$15,000,000	$25,000,000	$3,000,000	$180,000,000
T4	Kinder Morgan - Berth 410-411	9	$13,000,000	$40,000,000	$3,400,000	$90,000,000
HIO	Runway 12/30	12	$19,000,000	$66,000,000	$3,100,000	$66,000,000
Total			$870,800,000	$2,174,905,000	$105,800,000	$2,100,000,000
PORT	HQP2 - Port Administrative Offices	6a	$70,000,000	$109,600,000	$105,800,000	$2,100,000,000

The annual revenue and regional economic impact figures shown in the table for the PDX assets represent the portions of the total PDX revenue and regional impact assumed to be attributable to the specific functional asset group that was addressed in the study. The Central Utility Plant does not provide direct revenue, but its function supports the terminal and concourse with heating, cooling, and electrical power. Further, the CUP supports airfield assets with electrical power for lighting and navigational aids to enable night-time and poor weather operations. The P2 parking structure provides a significant revenue stream, but is not a primary component of minimum aviation operations. The

[2] The Local and Regional Economic Impacts of the Port of Portland, 2011, Martin Associates, March 15, 2012.

ground maintenance facilities and the ARFF serve as support in maintaining the daily operations of the key PDX assets.

The proportion of total PDX revenue and total regional impact to assign to the asset group was estimated by proportioning revenue and regional impact by the number of aircraft gates on Concourse C in comparison to the total number of aircraft gates at PDX. Forty percent of the gates at PDX are on Concourse C. Thus, 40% of the total annual $200 million Port revenue from PDX - $80 million – was assumed for the asset group. Similarly, 40% of the $3.9 billion PDX annual regional economic impact - $1.56 billion – was assumed for the asset group. The figures are believed to be conservative, as the productivity of functioning aircraft gates could be expected to increase with a fewer number of gates available. It should be noted that the existing Concourse C gates currently accommodate more passengers than the average of the total passengers traveling through PDX divided by the total number of gates. In 2014, Concourse C with 40% of boarding bridge gates at PDX accommodated approximately 50% of the total passengers traveling through PDX, and 60% of the passengers on aircraft served with boarding bridges. As noted previously, the figures in the table were used in the benefit-cost analyses of potential risk mitigation measures conducted in the study.

1.4 Report Organization

The Seismic Risk Assessment Study Final Report is organized, following this Introduction, as outlined below:

1. INTRODUCTION
2. SEISMIC SETTING AND HAZARDS

   Summarizes seismic conditions of the Portland area as considered in this study, and provides hazard analysis and code comparisons. Additional, detailed geotechnical information is contained in the geotechnical report in Appendix 2.

3. PERFORMANCE ASSESSMENT OF BUILDING ASSETS

   Summarizes the assessment of building assets, including building descriptions, results of preliminary performance assessments, and descriptions of potential measures to mitigate seismic risk for selected buildings. A detailed report on the building assessments is contained in Appendix 3.

4. PERFORMANCE ASSESSMENT OF MARINE ASSETS

   Summarizes the assessment of marine assets, with facility descriptions, results of performance assessments, and descriptions of potential mitigation measures. A detailed report on the assessment of marine facilities is contained in Appendix 4.

5. PERFORMANCE ASSESSMENT OF RUNWAY ASSETS

   Summarizes the assessment of runway assets, with descriptions of soil conditions, discussion on target performance criteria, estimates of runway downtimes, and descriptions of potential mitigation measures

6. SEISMIC RISK AND BENEFIT-COST ANALYSES

   Describes the seismic risk analysis for the assets studied – buildings, marine facilities, and runways, describes the methodology for the benefit-cost analysis, and presents the benefit-cost findings,

7. CONSIDERATIONS FOR CRITICAL NON-PORT ASSETS AND LIFELINES

   Provides a brief outline of critical assets and services that are not owned by the Port but on which the Port's operations are dependent, for future study.

8. CONCLUSIONS AND RECOMMENDATIONS

   Presents conclusions and recommendations of the study, with recommendations for five specific, prioritized projects, a brief discussion of suggested next steps, and several general recommendations.

APPENDICES

Appendix 1 – Study Team

Provides information on the Consultant study team.

Appendix 2 – Geotechnical Report

Contains the geotechnical consultation report prepared for the study, which includes background on geotechnical and seismic conditions, describes the analysis approach, and describes soil conditions and site-specific soil response estimates.

Appendix 3 – Seismic Assessment of Building Assets

Contains the detailed report on seismic assessments of the building assets considered in the study.

Appendix 4 – Marine Facilities Seismic Vulnerability Assessment

Contains the detailed report on seismic assessments of the marine assets addressed in the study.

Appendix 5 – Seismic Risk and Benefit-Cost Analyses

Contains the full report documenting the seismic risk and benefit-cost analyses conducted in the study.

Appendix 6 - Dependency of the Port of Portland on Regional Lifelines and Utilities

Contains a report presenting a general discussion and illustration of the exposure of Port assets to natural and man-made hazards, and the dependency of the Port's operations on regional lifeline networks. The report includes a suggested approach for assessing lifeline vulnerability.

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2. SEISMIC SETTING AND HAZARD LEVELS

The following summarizes seismic conditions of the Portland area as considered in this study, and presents hazard analysis and code comparisons. More detailed information regarding the geotechnical engineering aspects of the study is can be found in the geotechnical report prepared by Geotechnical Resources, Inc. and New Albion Geotechnical contained in Appendix 2.

2.1 General

On a regional scale, the Port of Portland facilities lie within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely, et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range mountains to the west and the Cascade Range mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The majority of Port facilities lie approximately 95 km inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the over-riding North American Plate, as shown on Figure 2.

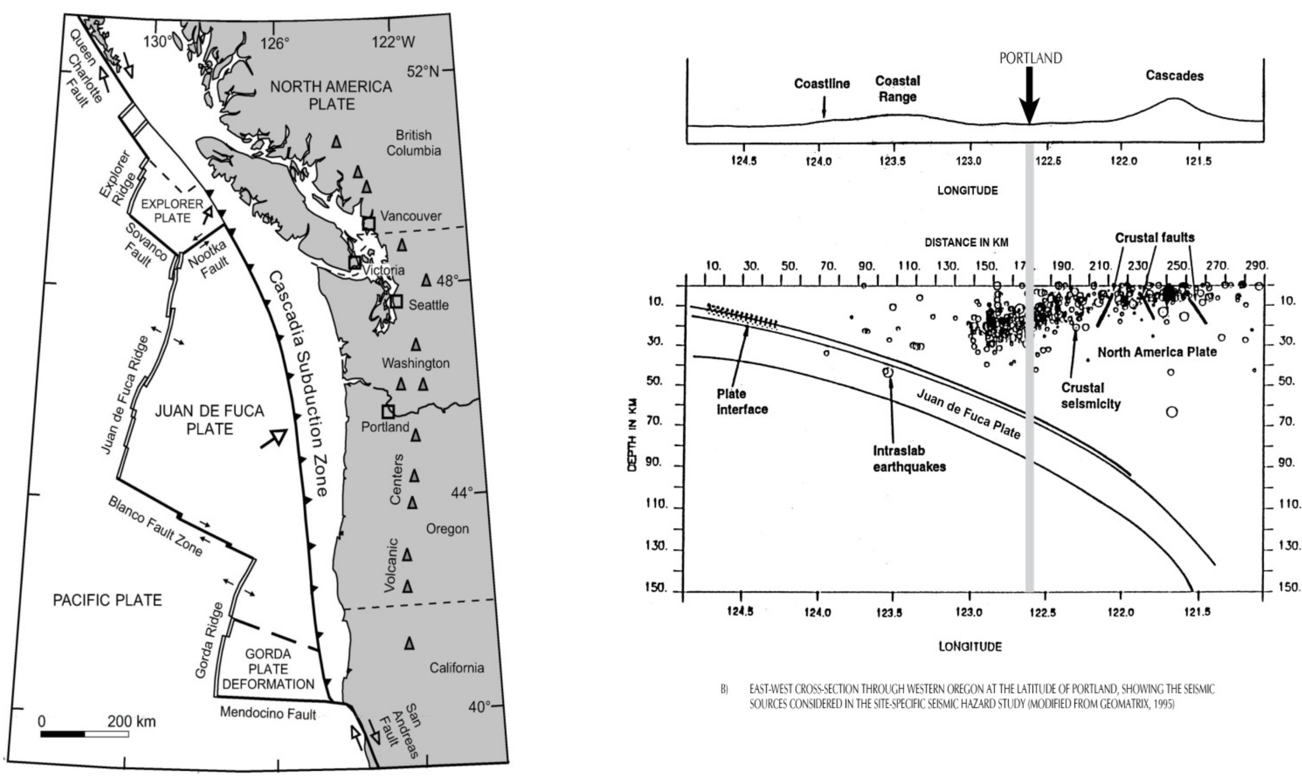

Figure 2 – Tectonic Map of the Pacific Northwest

Figure 2 also shows an east-west cross-section through western Oregon at the latitude of Portland depicting the historic seismic sources considered in the site-specific seismic hazard analysis of this study.

On a local scale, the majority of the Port of Portland facilities lie within the Portland Basin, a large, well-defined, northwest-trending structure characterized as a right-lateral pull-apart basin in the forearc of the CSZ. The Portland Basin is bounded by high-angle, northwest-trending, right-lateral strike-slip faults that are considered to be seismogenic; however, the relationship between specific earthquakes and individual faults in the area is not well understood since few of these faults are expressed clearly at the ground surface. A limited number of intra-basin faults have been mapped on the basis of stratigraphic offsets and geophysical evidence. The Port sites are located in relatively close proximity to the inferred traces of the Portland Hills Fault and the East Bank Fault indicated on published geologic mapping (Personius, et al., 2003) and shown on Figure 3. The fault locations on the geologic map are inferred or approximate. Other faults may be present within the basin, but clear stratigraphic evidence regarding their location and extent is not presently available. Appendix 2 contains more detailed information.

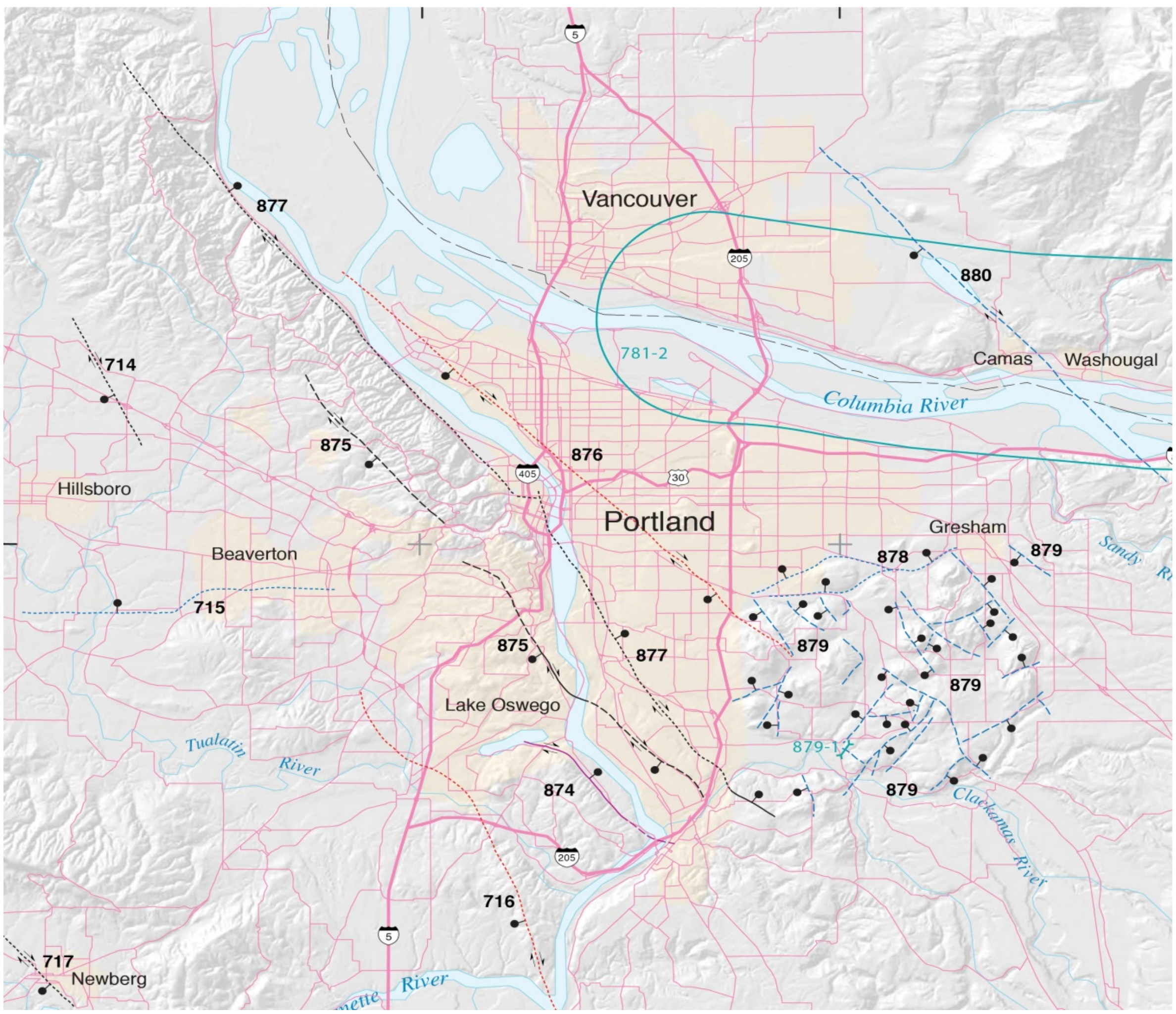

Figure 3 – Tectonic Map of the Portland Area

2.2 Seismicity Overview

Based on the seismic setting described above and information presented at recent workshops and conferences (United States Geological Survey (USGS), 2012a, b), the potential seismic sources that may affect the geographical area can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate; *deep, subcrustal (intraslab) events* related to deformation and volume changes within the subducted mass of the Juan de Fuca plate; and *local crustal events* associated with movement on shallow faults within and adjacent to the Portland Basin. Probabilistic and deterministic seismic hazard evaluations for these sources as related to this study are discussed in the following.

2.3 Probabilistic Hazard

The ground motions and seismic load levels currently adopted by building codes are primarily based on a probabilistic seismic hazard analysis (PSHA) using the 2008 USGS seismic hazard database. The input for a PSHA consists of three significant components.

1) Identification of earthquake sources, locations, and physical characteristics (e.g., dip angle, rupture width, and length).
2) Characterization of the seismicity rate for each seismic source using an appropriate model (e.g., exponential or normal distribution).
3) Selection of empirical attenuation relationships that describe how the characteristics of the strong ground motions change as the waves propagate from the seismic source to a given site location.

These components all include aleatory and epistemic uncertainties associated with our limited knowledge and understanding of the fault sources and their predicted behavior. The PSHA method combines the uncertainties associated with these three components to obtain a probabilistic ground motion, which is defined by the likelihood of an earthquake of a specific magnitude occurring within a specific length of time

Table 2 on the following page summarizes the peak bedrock acceleration (PBA), 0.2-second (S_s) and 1-second (S_1) spectral accelerations for the uniform, or cumulative, hazard as a comparison at the 72-, 225-, 475-, 975-, and 2,475-year return periods selected for the probabilistic portion of this study. Consistent with the discussion above, these probabilistically-based values include contributions from crustal, subcrustal, and subduction zone motions. The geotechnical report in Appendix 2 includes additional detail on how these bedrock level ground motions are adjusted for site effects and utilized for liquefaction and lateral spreading analyses.

Table 2 – Probabilistic Ground Motions

Probability	Return Period, years	USGS 2008 Database: Peak Bedrock Acceleration, PBA, g	0.2-second Spectral Acceleration, S_s, g	1-second Spectral Acceleration, S_1, g
50% in 50 years	72	0.05	0.11	0.03
20% in 50 years	225	0.13	0.28	0.09
10% in 50 years	475	0.20	0.46	0.16
5% in 50 years	975	0.29	0.67	0.29
2% in 50 years	2,475	0.44	1.02	0.40

2.4 Deterministic Analysis

The study conducted a deterministic analysis of the Hillsboro Airport Runway 13/31, as opposed to the probabilistic analysis undertaken for PDX building assets. A M9.0 CSZ earthquake was selected for the deterministic assessment of the HIO runway. The peak bedrock motions associated with this earthquake scenario were estimated using the weighted Ground Motion Prediction Equations (GMPE) recommended by the USGS (2008) with an average bedrock shear wave velocity of 760 m/s. The following table summarizes the deterministic earthquake analysis with mean peak bedrock acceleration (PBA) for this CSZ scenario.

Table 3 – CSZ Deterministic Ground Motion

Source	Magnitude, M_w	Distance, R, km	Mean Peak Bedrock Acceleration, PBA, g
Cascadia Subduction Zone	9.0	80	0.17

The geotechnical report contained in Appendix 2 provides additional detail on how these deterministic earthquake parameters were used to evaluate the potential for liquefaction and develop settlement estimates for the HIO runway.

2.5 Marine and Building Code Considerations and Hazard Level Comparison

The probabilistic and deterministic scenarios discussed above were developed to bracket the range of hazard levels typically considered in both marine and building code applications for new and retrofit considerations. The following sections summarize the design-level earthquakes outlined in the applicable codes. These definitions and background are provided for comparison only, as the majority of this study focused on the full range of probabilistic hazard levels discussed in Section 2.3 above.

2.5.1 Marine Code Comparison

The recently released American Society of Civil Engineers (ASCE) standard, *Seismic Design of Piers and Wharves* (ASCE SDPW, 2014), is a performance-based design standard for new non-public piers and wharves and summarizes common hazard levels considered for at various design levels. This standard defines ground motions for three seismic hazard levels, with those used for a "high" design level as described below: the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE).

> OLE is defined by 50% probability of exceedance in 50 years, which corresponds to ground motions with an expected recurrence interval of 72 years and represents a performance level with minimal structural damage.
>
> CLE is defined by 10% probability of exceedance in 50 years, which corresponds to ground motions with an expected recurrence interval of 475 years, and represents a performance level of controlled and repairable structural damage.
>
> DE is defined per ASCE 7-05, which develops the response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE). The MCE is generally represented by a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years), except where subject to deterministic limitations (Leyendecker et al., 2000). The design-level response spectrum that represents the DE is obtained by taking two-thirds of the MCE level ground motions.

2.5.2 Building Code Comparison

The seismic evaluation and retrofit of structures is commonly accomplished in accordance with ASCE 41-13, *Seismic Evaluation and Retrofits of Existing Buildings*, which also references ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-10 is the reference document for the recently-adopted 2014 Oregon Structural Specialty Code. The ASCE 41-13 requirements specify evaluation of the seismic hazard based on four hazard levels, referred to as: BSE-1N, BSE-2N, BSE-1E, and BSE-2E. These hazard levels represent the Basic Performance Objective Equivalent to New Building Standards (BPON) and Basic Performance Objective Equivalent to Existing Buildings (BPOE).

The BSE-1N and BSE-2N hazard levels, which are intended to meet the requirements of BPON, are similar to the previous ASCE 41-06 hazard levels BSE-1 and BSE-2, except that BSE-1N and BSE-2N are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motions consistent with ASCE 7-10. The BSE-2N seismic hazard level is defined by the MCE_R ground motions, and thus can be considered as the seismic hazard with 2% probability of exceedance in 50 years multiplied by a risk coefficient (i.e., represents a targeted risk level of 1% in 50 years for probability of collapse). It should be noted that MCE_R also incorporates adjustments from "geomean" ground motions to "maximum-direction" ground motions. The adjustment to obtain "maximum direction" values from "geomean" values requires applying a factor of 1.1 to the short-period spectral values and 1.3 to the long-period spectral values. The BSE-1N hazard level is provided in ASCE 41-13 with the intent of matching the design earthquake ground motions in ASCE 7-10, and therefore is defined as two-thirds of the BSE-2N hazard level.

The BSE-1E and BSE-2E are the two seismic hazard levels intended to meet requirements of the BPOE. The BSE-1E and BSE-2E hazard levels are defined by an earthquake having a probability of exceedance of 20% in 50 years and 5% in 50 years, respectively, in the maximum direction of shaking. The ASCE 41-13 guidelines require capping the BSE-1E and BSE-2E spectral values at the BSE-1N and BSE-2N values, respectively, presuming the seismic design parameters for existing buildings should not be greater than that of new buildings.

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2014 Oregon Structural Specialty Code

3. PERFORMANCE ASSESSMENT OF BUILDING ASSETS

The following narrative is a summary of the performance assessments of the selected key Port building assets. The narrative provides a brief description of each building, presents the results of preliminary performance assessments, and describes potential measures to mitigate the risk of earthquake damage for a group of the key building assets. The potential risk mitigation measures were used in the benefit-cost analyses. Comprehensive documentation of the building assessments is contained in the full building structural report prepared by KPFF Consulting Engineers in Appendix 3.

3.1 Background and General Performance Assessment

Review of building assets involved site observations, study of drawings of the existing construction, and limited analysis assessing the expected building performance in specific earthquake scenarios. Earthquake scenarios took into account site-specific geotechnical conditions and seismic response spectra. The buildings have been constructed over a 60-year period, with different materials and structural systems and under different building codes. Some of the buildings have undergone renovations in later projects, some more than once. As a result, the buildings have varying degrees of seismic deficiencies and vulnerability. Table 4 on the following page summarizes qualitatively the general conditions for each of the building assets considering the factors outlined below in relation to the current Oregon Structural Specialty Code (OSSC).

- Liquefaction – Most of the buildings have slab-on-grade base levels, and will suffer effects from seismically-induced soil settlement. Settlement will impact not only the slab itself, but any MEP, baggage handling, or other systems supported by the slab. Older piles do not appear to penetrate the denser sand layers adequately and may experience settlements of several inches, which will be non-uniform from column to column. This differential settlement will distress the structure and any rigid non-structural elements. Buildings with spread footings will experience severe deformations from settlements.
- Lateral Force Resisting System (LFRS) Force Capacity – Older structures that have not been seismically upgraded are deficient in strength to resist the current Code-prescribed seismic forces. Newer structures, or those that have been upgraded, generally have adequate strength for the Code level performance objective; however, the structures may be deficient for the higher performance objective of Immediate Occupancy at a 475-year return period event.
- LFRS Ductility – Many of the structures are deficient with respect to current Code requirements for ductility and have structural systems that are either no longer permitted for new construction in this seismic region, and/or are penalized in more recent codes for their lack of ductility; the penalty takes the form of requirements to design for significantly greater loads. Ductility includes concrete shear wall boundaries, brace connections, braced frame columns, drag connections, diaphragm connections, and similar items.

Table 4 – Existing Building Condition in Relation to Current OSSC Code

Building	Year Built	Seismic Upgrade?	Liquefaction		LFRS	
			Foun-dations	Slab-on-grade	Force Capacity	Ductility
1 – CUP	1972, 1992, 1994	1998	Short Piles			
3 - Concourse C	1999	NO	Long Piles			
4 T1 - Terminal Ticket Lobby	1973, 1996	1996	Short Piles			
4 T2 - Terminal South Node	1999	NO	Long Piles			
4 T3 - Terminal OMP South	1956, 1986, 2002	2002	Mixed Piles			
4 T4 - Terminal OMP Central	1956	2006	Short Piles			
5 - ARFF	1996	NO	Mat			
6a - HQ/P2 (North)	2007	NO	Long Piles			
6b - P2 (South)	2007	NO	Long Piles			
13 - PDX Ground Maint. Facility B	1982	NO	Spread			
14 - PDX Ground Maint. Facility A	1982	NO	Spread			
15 - PDX Ground Maint. Facility C	1982	NO	Spread			
16 - T6 Maintenance Workshop	1973	NO	Spread			
17 - T6 Electrical Shop	1982	NO	Spread			

Legend

Good
Fair
Poor

3.2 Performance Assessments of Building Assets

Descriptions of the building assets and findings of the preliminary performance assessments, as summarized in Table 4, are presented below. The discussion includes comparisons of design code forces versus current code forces, evaluations using performance criteria in ASCE 41 for selected assets, and estimates of downtime. The downtime estimates are based on engineering judgment of the time required to repair/rebuild a structure to an occupiable condition after a 475-year return period earthquake, considering the type of structure, expected resilience, and effects of liquefaction. The estimates do not include time impacts of procuring funding or permits, design and procurement time, availability of contractors or construction supplies, or disruption to utilities outside the building, vehicular access to PDX, Port communications and personnel issues, and similar concerns.

More detailed discussion on the assessment of building assets can be found in Appendix 3.

3.2.1 Central Utility Plant

Description

The Central Utility Plant is a steel-frame structure with concrete over metal decking at the second floor and metal deck roofing. The ground floor is a structured concrete slab on grade designed for high gravity loads. The lateral system is composed of a combination of steel braced frames, wire rope braced frames, and steel truss moment frames. The building is supported on a pile foundation, with a combination of timber piles from the original construction and auger-cast concrete piles from subsequent expansions and upgrades. The piles are relatively short. The CUP was originally constructed in 1970, expanded in 1992, and seismically upgraded to the 1994 Uniform Building Code (UBC) in a later project. The seismic upgrade was intended to achieve a Hazardous Facility Importance Factor of 1.25.

Seismic Performance Assessment

The 1994 UBC to which the CUP was upgraded specified design forces that are between 65% and 87% of current code design forces for the systems used. The lack of ductile detailing in the ordinarily braced frame is expected to result in significant localized damage. Additionally, the stiffness disparity of the structural systems is likely to result in undesirable distribution of lateral forces. Cable braces and moment frames may experience deflections exceeding those desirable for rigid building attachments to stiff exterior masonry work and glazing systems. The exterior masonry walls are particularly susceptible due to the thinness of the sections and the lack of ductility.

Mechanical, electrical, and plumbing (MEP) and systems equipment are seismically braced and are expected to perform satisfactorily for an essential-facility level of design, except for the effects of excessive deflections of the building structure near rigid connections to equipment and the effects of exterior ground settlement due to liquefaction where rigid utilities enter the pile-supported buildings. Parts storage racks at some locations do not appear to be seismically braced. However, damage to these racks would not appear to compromise operations of the CUP.

Through evaluation of the existing CUP structure using ASCE 41 performance criteria and site-specific geotechnical information, the return-period ground motions for the three performance levels addressed in the study, without consideration of liquefaction effects, are estimated to be as listed below. The estimates mean that the structure would be expected to perform at the level indicated in ground motions up to the levels of the return period shown.

Immediate Occupancy:	110 years
Life Safety:	270 years
Collapse Prevention:	680 years

Soils near the ground surface at the CUP site, similar to most of the sites of Port assets, are susceptible to liquefaction during a seismic event. The relatively short piles penetrate the more stable underlying denser sand layer by only about five feet. The shallow penetration of the denser sand layer is not likely to be sufficient to prevent liquefaction-induced settlement of several inches. The settlement would cause stresses throughout the structure, and particularly in the exterior wall elements and MEP connections. The structured slab-on-grade would mitigate the effects of the larger free-field soil settlement to some extent, but not to an Immediate Occupancy level.

Approximately 80 feet of corrugated steel pipe (CSP) utility tunnel exists between the CUP and the utility tunnels under the HQ/P2 buildings. While the CUP and the utility tunnels under the HQ/P2 buildings are pile-supported, the CSP tunnel is not. The CSP tunnel will settle in the event of liquefaction, resulting in distress and likely damage to the tunnel and utilities at the connections to the CUP and the tunnels under the HQ/P2 buildings.

Downtime for the CUP to rebuild and repair the building to a usable condition following a 475-year event is estimated to be approximately 12 months. Additional time could be needed to procure, install, and commission replacement equipment.

3.2.2 PDX Concourse C – Building Sections C1, C2, C3

Description

Concourse C is generally a two-story building with mechanical penthouses on the roof and a below-grade utility tunnel. A third story on the east part of the concourse is occupied by the PDX Emergency Operations Center. The building is composed of three separate structures of similar construction separated by seismic joints. Figure 4 on the following page illustrates the composition of the building sections.

The building construction is steel framing with composite concrete decks; the lateral system consists of steel special moment-resisting frames. Exterior walls are constructed of masonry block on the lower level and metal panels and glazing on the upper levels. The building is supported on steel piles. The ground floor is slab-on-grade with integral grade beams. The concourse was constructed in the late 1990s.

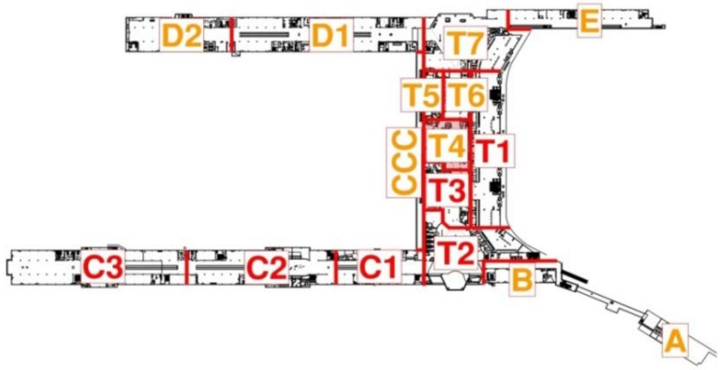

Figure 4 – Terminal and Concourse Building Layout

Seismic Performance Assessment

Concourse C was designed under the 1994 Uniform Building Code (1996 OSSC), but used the more ductile "dogbone" moment frame detailing that was new at that time. Detailing is generally consistent with current code for Seismic Occupancy Category III, representing enhanced life-safety criteria between standard occupancy and Essential Facility. The base shear design is 103% of current code design forces. Lateral drifts of the building in 475-year ground motions will exceed current detailing for drift of exterior components, seismic joints, and interior architectural components and utilities for Immediate Occupancy. The exterior metal panel system is flexible, but damage to glazing is expected.

The pile foundations are sufficiently deep to limit settlement of the building to no more than approximately one inch in a long return period earthquake. However, liquefaction at the site could result in ground settlements of approximately eight inches at the 475-year event and potentially more than a foot at the 1000-year return period event. The ground settlement would cause significant distress in the slab-on-grade and the utility tunnel which rely on soil support. Architectural elements and MEP equipment and systems supported by the slab-on-grade and the utility tunnel would similarly experience substantial distress.

Evaluation of the existing structure per ASCE 41 performance criteria and using site-specific soil response spectra, without consideration of liquefaction effects, produces the estimates of return period ground motions listed below for the three performance levels considered in the study. The evaluation represents all three sections of the concourse.

Immediate Occupancy:	130 years
Life Safety:	1000 years
Collapse Prevention:	2200 years

Bracing of MEP equipment and systems are expected to perform satisfactorily for an enhanced life-safety level design, except where affected by settlement of the slab-on-grade and the utility tunnel resulting from soil liquefaction. Immediate Occupancy performance will be limited as indicated by the low return period. This is a result of potential permanent building offset that could occur due to yielding of the moment frames, along with anticipated damage to glazing and the MEP systems caused by building drift due to the relatively flexible moment-frame system.

Downtime to restore Concourse C to an occupiable condition following a 475-year return period event is estimated to be approximately 2 months.

3.2.3 Terminal Ticket Lobby – T1

Description

The terminal building is composed of seven different building units, as depicted in Figure 4. The Terminal Ticket Lobby, unit “T1”, is the eastern portion of the terminal building and contains the central section of the ticketing lobby on the upper level and baggage claim on the lower level. The building unit has a narrow mechanical mezzanine along the west side. The gravity structural system is composed of steel framing with non-composite concrete on metal deck at the upper level and open-web steel joists with metal deck at the roof. The foundations are steel piles, and the ground floor is slab-on-grade. The building was originally constructed in 1973, and was modified by the Terminal Access Program project in the mid-1990s. The modifications included seismic upgrades consisting of concrete shear walls and steel braced frames.

Seismic Performance Assessment

Upgrades to the T1 building in the mid-1990s were designed under the 1991 UBC. Design forces under the 1991 code were below current design force requirements. The base shear design for concrete shear walls is 97% of current code design forces, and for steel concentric braced frames is 63% of current code. The building was not constructed with ductile detailing as would be required under current code. The lack of ductile detailing in braced frames and shear walls is expected to result in substantial localized damage in a significant earthquake.

Original steel piles are not deep enough to penetrate the denser sand substrate adequately to prevent settlement of the building structure. The piles could settle several inches in a significant seismic event, leading to distress in the building framing and floor systems along with damage to the secondary structural systems. Micropiles were installed in the seismic upgrade in the mid-1990s to support the new shear walls. The micropiles are deep enough to prevent significant settlement, which could result in these piles becoming overloaded as the adjacent older piles settle and loads are transferred. Soil liquefaction could cause settlement of the slab-on-grade of a foot or more, leading to damage to architectural and MEP elements supported by the slab. Lower level exit vestibules would also be damaged by settlement.

Evaluation of the existing structure in accordance with ASCE 41 criteria shows the following estimates of return period ground motions for the three performance levels, not considering the effects of liquefaction:

Immediate Occupancy:	600 years
Life Safety:	920 years
Collapse Prevention:	1900 years

Bracing of new MEP equipment and systems would likely perform satisfactorily for an enhanced life-safety level design, except where affected by liquefaction-induced settlement of the slab-on-grade. Older equipment installed prior to newer bracing requirements would be considerably more vulnerable.

Downtime to restore T1 to an occupiable condition following a 475-year event is estimated to be approximately 12 months.

3.2.4 Terminal South Node – T2

Description

The south node of the terminal, unit "T2", is a three-story structure at the junction of the terminal building, Concourse B, and Concourse C constructed in the late 1990s. The building accommodates baggage systems on the ground level, the south security checkpoint and various food and retail concessions on the second level, and offices on the third (mezzanine) level. The structural system is steel framing with composite concrete decks on the second and third levels, and reinforced concrete bearing and shear walls for the lateral system. The building has a steel pile foundation, and the ground floor is slab-on-grade.

Seismic Performance Assessment

Building T2 was designed to the 1994 UBC. The base shear design force is 103% of current code requirements. However, detailing of shear wall reinforcing does not meet current code requirements, notably for ductility. The lack of ductile detailing in the shear walls could result in localized damage in a significant earthquake. The pile foundations are deep and are expected to settle no more than approximately one inch in a long return period earthquake. Liquefaction of the soil could cause settlement of the slab-on-grade of approximately 10 inches in the 475-year event and more than a foot in the 1000-year event. The settlement would affect all equipment and systems that are supported by the slab-on-grade.

Return period ground motions estimated for the three performance levels as produced by ASCE 41 evaluation, without consideration of liquefaction, are as follows:

Immediate Occupancy:	1700 years
Life Safety:	2500 years
Collapse Prevention:	>2500 years

Bracing of MEP equipment and systems would likely perform satisfactorily for an enhanced life-safety level design, except where affected by liquefaction-induced settlement of the slab-on-grade.

Downtime to restore the Terminal South Node to an occupiable condition following a 475-year event is estimated to be approximately 2 months.

3.2.5 Terminal Oregon Marketplace (OMP) South – T3

Description

The Terminal Oregon Marketplace South, unit "T3" on Figure 4, is a three-story combination of structures accommodating baggage handling and various terminal functions on the ground level, food and retail concessions on the second level, and offices and mechanical equipment on the third, mezzanine level. The original structure was built in 1956 of one-way concrete slab and beam construction. The building was expanded and upgraded in 1986 and 2002 with steel and composite deck construction. The various structures have been connected and seismically upgraded to behave as a single structure. The lateral system is a combination of concrete shear walls and steel concentric braced frames. The building is supported on a combination of steel piles and micropiles. The ground floor is slab-on-grade.

Seismic Performance Assessment

Seismic upgrade of Terminal OMP South was started under the 1994 UBC and completed under the 1997 Code. Base shear design force for concrete shear walls is 107% of current code requirements; for steel concentric braced frames, the base shear design force is 70% of current code requirements. Ductile detailing in the braced frames, shear walls, and drag connections does not meet current code. The lack of ductile detailing could result in greater localized damage than would be expected with current detailing.

Older steel piles do not appear to be deep enough to penetrate dense sand adequately to prevent settlement of the building structure. As noted for T1, these shallow piles could settle several inches in a significant seismic event, causing distress in the old concrete building framing and floor systems along with damage to the secondary structural systems. Micropiles were installed in the seismic upgrade to support the new shear walls. The micropiles are deep enough that they are not expected to settle significantly. As noted for T1, this could lead the micropiles to become overloaded as the adjacent older piles settle and loads are transferred. Soil liquefaction could cause settlement of the slab-on-grade of a foot or more, leading to damage to architectural and MEP elements supported by the slab.

Evaluation of the existing structure per ASCE 41 performance criteria and site-specific response spectra produces the following estimates of return period ground motions for the three performance levels, not considering the effects of liquefaction:

Immediate Occupancy:	140 years
Life Safety:	1400 years
Collapse Prevention:	2500 years

Bracing of new MEP equipment and systems would likely perform satisfactorily for an enhanced life-safety level design, except where affected by liquefaction-induced settlement of the slab-on-grade. Older equipment installed prior to newer bracing requirements would be considerably more vulnerable.

Downtime following a 475-year event to restore Terminal OMP South to an occupiable condition is estimated to be approximately 24 months.

3.2.6 Terminal OMP Central – T4

Description

Similar to Terminal OMP South, the Terminal Oregon Marketplace Central building, unit "T4", is a three-story structure accommodating baggage handling and various terminal functions on the ground level, food and retail concessions on the second level, and offices and mechanical equipment on the third, mezzanine level. The original structure was built in 1956 of one-way concrete slab and beam construction. A new roof structure was built of steel framing and composite concrete on steel deck over the western portion of the building in 1986. The structure has undergone partial seismic upgrade. The lateral system is a combination of concrete shear walls and steel concentric braced frames. Foundation support is provided by piles. The ground floor is slab-on-grade.

Seismic Performance Assessment

The Terminal OMP Central partial seismic upgrades have been designed for the 1997 UBC for force level. Within the lateral system of concrete shear walls and concentric braced frames, the older elements do not meet current code requirements for ductility detailing. The required seismic design force for steel frames has increased substantially since the existing frames were designed and constructed. Base shear design force for the concrete shear walls is 107% of current code requirements; for steel concentric braced frames, the base shear design force is 70% of current code requirements. The structure is expected to perform poorly in a major seismic event until the seismic upgrade is completed. The upgrade will need to address the existing lack of ductility to achieve better than marginal life-safety performance. Based on analysis using site specific response spectra developed in this study, collapse of the building could be possible at approximately the 1000-year return period ground motions. To improve this performance, the ductility of the structural system should be addressed as a whole when the seismic upgrade is completed.

The site is susceptible to liquefaction. Similar to other older buildings, the existing piles do not penetrate the dense sands adequately to prevent liquefaction-induced settlement, and the building could be expected to settle several inches in a significant earthquake. Resulting differential settlements would cause substantial damage to the older concrete structure; damage could be severe enough to require extensive repair or even reconstruction. The slab-on-grade ground floor would be expected to settle as much as 10 inches in a 500-year return period event, and more than a foot in larger earthquakes.

Similar to the other terminal buildings, bracing of new MEP equipment and systems would likely perform satisfactorily for an enhanced life-safety level design, except where affected by liquefaction-induced settlement of the slab-on-grade. Older equipment installed prior to newer bracing requirements would be considerably more vulnerable.

Downtime following a 475-year event to restore Terminal OMP Central to an occupiable condition is estimated to be approximately 24 months.

If this area of the terminal building is to remain intact in long-term plans for the terminal, completion of the remaining portions of the voluntary seismic upgrade should be a top priority. It should be noted that completing the upgrade could be required by the City of Portland if any substantial work is undertaken in the area, regardless of long-term plans, unless other agreements are made with the City.

3.2.7 PDX Aircraft Rescue and Fire Fighting (ARFF) Facility

Description

The PDX ARFF facility is single-story building with two partial mezzanines. The east portion of the building is occupied by office and living quarters for personnel; the west portion of the building shelters firefighting and rescue trucks. The building is constructed with concrete masonry bearing/shear walls. The lateral system consists of the masonry shear walls with combination steel moment frame/masonry shear wall piers at the truck bays. The roof is a combination of composite concrete on metal deck over the east part of the building and metal roof deck over the truck bays. The building foundation is concrete slab-on-grade with thicknesses of 18 to 24 inches under bearing walls and 12 inches in open floor areas.

Seismic Performance Assessment

The ARFF building was designed to the 1991 UBC as an Essential Facility with an Importance factor of 1.25. The base shear force for design is 107% of the current code requirement. While the ARFF building meets current code seismic design force for an Essential Facility, the structure likely does not meet steel reinforcing ductility requirements of the current code. A lack of ductile detailing could result in greater localized damage than would occur with the prescribed ductility, although the effects would be less pronounced in a shorter, stiffer building such as the ARFF facility.

The original design was based on a limited liquefaction analysis, consistent with geotechnical knowledge at the time. At the time the building was designed, minimal liquefaction settlement was expected with some greater potential for lateral spreading. Current knowledge indicates minimal likelihood of lateral spreading at the site, but high potential for significant liquefaction-induced settlements. Analysis suggests settlements of approximately six inches at a 200-year return period ground motions, and settlements greater than a foot at the 1000-year return period. The mat foundation can be expected to mitigate the effects of settlement to a degree. However, differential settlements could create problems with operating truck-bay doors and with vertical offsets between interior and exterior slabs.

MEP equipment and systems are generally braced and are expected to perform satisfactorily for an essential facility level design. As with significant settlement of any building, however, utility services and connections to building-supported utilities could be compromised.

The downtime estimated to restore the ARFF facility to an occupiable condition following a 475-year return period event is two months. A plan should be developed for extricating trucks from the building in the event that bay doors become inoperable as the result of settlement. Such a plan would help to ensure that appropriate tools are available and roles and actions of personnel are assigned.

3.2.8 Port Headquarters (HQ) and P2 Parking Structure North and South

Description

HQ/P2 is composed of the P2 parking structure and the Port Headquarters office building above the north part of the parking structure. The parking structure consists of seven floors of public parking; the office building is three floors. The parking structure is constructed of post-tensioned concrete beams and floor slabs. The lateral system in the garage is comprised of special concrete shear walls with special concrete moment resisting frames in the north-south direction and special moment resisting frames in the east-west direction. The ground floor is concrete slab-on-grade. The office levels are constructed of steel framing with composite concrete over metal deck slabs. The lateral system for the office levels consists of special steel moment resisting frames in both directions. The building is supported on long pile foundations.

Seismic Performance Assessment

HQ/P2 was designed under the 2006 International Building Code (2007 OSSC). Base shear design forces are 100% of the current code requirements. The north part of the building consisting of the parking structure with the office building, HQ/P2 North, meets current code design and detailing requirements for structured standard parking and office space. Code performance expectations are for collapse prevention in a 2475-year return period ground motions event, and life-safety for an earthquake with accelerations of two-thirds the accelerations of the 2475-year event. P2 South, the south part of the building consisting of just the parking structure, exceeds the code performance expectations.

The site is susceptible to liquefaction as are most areas of PDX. The long piles will prevent significant settlement of the building in a long return period earthquake. However, the slab-on-grade could settle significantly; settlement could potentially be a foot or more with a 1000-year return period event. The settlement would cause disruption of functions and services that are supported by the slab-on-grade. MEP equipment and systems are braced in accordance with current code and will perform satisfactorily except where supported by the slab-on-grade.

The utility tunnel between the Central Utility Plant and the terminal lies under P2 South. The tunnel is pile supported and is not expected to be significantly affected by liquefaction and settlement.

Downtime to restore HQ/P2 to an occupiable condition following the 475-year return period event is estimated to be approximately one month. Complete repairs, particularly of the slab-on-grade, would take a longer time.

3.2.9 PDX Ground Maintenance Facilities (Buildings A, B, and C)

Description

Three separate buildings of the PDX Ground Maintenance Facilities complex were evaluated. The three buildings consist of the Administration and Shops Building ("Building B"), a vehicle parking and storage warehouse ("Building A"), and a vehicle maintenance and supply storage warehouse ("Building C"). Building B is mostly a single-story building housing office, storage, and maintenance spaces. There is a small second-story area with a balcony and two interior mezzanines. A large soil berm is built up against the south side of the building, rising to approximately mid-height of the wall. Building A is a large, open, single-story structure. A soil berm exists along the east face of the building, and is built up to approximately mid-height of the wall similar to the berm at Building B. Building C is generally a single-story structure, but contains a small interior mezzanine.

The three buildings were built in the 1980s and are of similar construction. Precast, hollow-core concrete panels form exterior walls and serve as both exterior load-bearing elements and shear walls. At Buildings B and A, precast panels at the locations of the soil berms are supported by cast-in-place concrete retaining walls. At Building B, the roof structure is either wood framing with plywood diaphragms spanning up to 140 feet, or open web three-dimensional Unistrut trusses with steel roof decking. The roof system of Buildings A and C is composed of plywood sheathing over solid wood joists and glulam beams. The roof span between shear walls at Building A is 180 feet; at Building C, the span is 140 feet. Foundations at each of the buildings are spread footings, and ground floors are slab-on-grade.

Seismic Performance Assessment

The Ground Maintenance Facility buildings were designed under the 1979 UBC (1980 OSSC). The base shear design force for each building is 31% to 37% of current Code requirements. Lateral systems lack ductility. Roof diaphragms lack continuous cross-ties and the ledgers are loaded in cross-grain bending, both of which lead to common failure mechanisms in buildings of this type and era. Additionally, the roof levels of Buildings B and C are offset vertically, which can contribute to significant localized damage to structures during ground shaking.

The site is susceptible to liquefaction. Ground settlements of as much as 18 inches could occur with ground motions with a return period as short as 200 years. Spread footings could experience an additional foot of settlement. Settlements of this magnitude, along with the other seismic deficiencies, will likely result in the buildings being unusable after an event with a return period of approximately 200 years.

MEP equipment and systems appear to be generally unbraced, although most of the equipment and systems are minor in nature. With liquefaction-induced settlements, underground utilities and their connections to the buildings will likely be compromised.

Downtime to reconstruct the buildings after 200-year return period event is estimated to be approximately 16 months.

3.2.10 Terminal 6 Maintenance Warehouse

Description

The Maintenance Warehouse at marine Terminal 6 is a Butler-type, prefabricated steel building. The building consists of a one-story warehouse and shop space with two mezzanines, and a two-story office space.

The building structural system is comprised of steel framing with precast concrete wall panels. The roof is framed with z-girts and metal roof deck. The upper side walls are framed with Z-girts and metal panels. The two interior mezzanines have reinforced masonry walls; a two-story infill portion consists of full height reinforced masonry walls up to the roof level. The framing of the mezzanines and second story are open-web steel joists supporting concrete topping over metal decking. The lateral system is comprised of two bays of tension-only steel rod bracing along each exterior building wall, along with moment frames in the east-west direction. The ground floor is slab-on-grade and foundations are spread footings.

Seismic Performance Assessment

The Maintenance Warehouse was designed under the 1970 UBC. The design was based on wind loading. The design wind load is 77% of the seismic design load requirements in the current code in the east-west direction, and 35% of the seismic design load requirements in the north-south direction. The rod bracing and precast panels in the north-south direction lack the ductile configuration and detailing required by current code. The moment frames in the east-west direction also lack current code ductility, although the moment frames would be expected to perform better than the tension-rod bracing. Additionally, the site is susceptible to liquefaction during a seismic event. Liquefaction-induced settlements could exceed one foot at relatively short return period ground motions (less than 300 years). The structure and foundations are not designed to accommodate this magnitude of settlement, so extensive damage will occur from the liquefaction alone.

MEP equipment and systems appear to be unbraced, but are mostly of a relatively minor nature. Liquefaction-induced settlements will likely compromise underground utilities and their connections to the building.

Downtime following a 475-year return period event is estimated to be 12 months, which is the time estimated to reconstruct the building.

3.2.11 Terminal 6 Electrical Shop

Description

The Terminal 6 Electrical Shop is a Butler-type, prefabricated building consisting of a single-story warehouse and shop space with a mezzanine and a two-story office space.

The building is a steel-framed structure with roof and wall framing consisting of steel Z-girts and metal panels. The lateral system is comprised of steel moment frames in the east-west direction and two bays of tension-only cable bracing on each side of the structure in the north-south direction. Ground floor is slab-on-grade and foundations are spread footings.

Seismic Performance Assessment

The Electrical Shop was designed under the 1985 UBC (1986 OSSC), based on wind loading. The design wind load is equal to 167% of the seismic design load of current code requirements in the east-west direction and 28% of the seismic design load in the north-south direction. The rod bracing in the north-south direction lacks the ductile configuration and detailing required by current code. The moment frames in the east-west direction also lack current code ductility, but appear to have a significant excess capacity relative to design force. The full-height concrete masonry walls do not appear to have proper attachments to the roof and walls, or isolation. This condition could result in damage and falling debris hazards. Additionally, as noted for the Maintenance Warehouse, the site is susceptible to liquefaction during a seismic event, with settlements exceeding a foot at relatively short return period ground motions (less than 300 years). The structure and foundations are not designed to accommodate settlements of this extent, and extensive damage will result from the liquefaction alone.

A number of storage cabinets and parts racks inside the building were observed to not be seismically braced. Loose cabinets and racks can pose a hazard during a seismic event.

MEP equipment and systems are generally unbraced, but the equipment and systems are generally relatively minor. Liquefaction-induced settlements can be expected to compromise underground utilities and their connections to the building.

3.3 Mitigation Measures Considered for Selected Building Assets

Potential measures to mitigate seismic vulnerability were identified for a group of the PDX building assets evaluated in the study. The particular assets for which mitigation measures were explored consisted of the CUP, Concourse C, and three sections of the main terminal T1, T2, and T3. As considered in the study, the goal of the mitigation is to improve building performance to achieve conditions of immediate occupancy for the 475-year return period event. The potential mitigation measures are outlined in the following. More detailed information, including diagrams illustrating conceptual mitigation actions, is contained in Appendix 3.

It should be noted that the mitigation concepts presented in this report are based on limited engineering analysis supported by engineering judgment. In all cases, next steps should necessarily entail detailed geotechnical, structural, and MEP investigations and analyses, to more fully understand existing conditions and seismic vulnerabilities and to verify and refine or adjust the findings of this study. Among the elements to be investigated in detail are site specific liquefaction potential, site specific response spectra, structural capacity, and MEP systems that are critical for Immediate Occupancy.

3.3.1 Central Utility Plant Mitigation

Successful mitigation of seismic risk at the CUP, and at other building assets, must address both the soil liquefaction potential and the seismic strength and ductility of the lateral force resisting system (LFRS). Additionally at the CUP site, mitigation should address the existing utility tunnel. Potential mitigation possibilities are identified below. Replacement of the CUP is not specifically addressed as an alternative in this report. However, the Port may wish to consider replacement of the facility as among the potential measures that could be taken to mitigate the risk.

Liquefaction

The risk of settlement caused by liquefaction could be mitigated by adding pile support to the existing foundation. This could be accomplished by installing micropiles at each building column and other major load-bearing element, and at locations of critical equipment inside the building and in the equipment yard. Detailed geotechnical and structural studies to more fully understand expected settlements and resulting effects on the foundations, slab-on-grade, and other building systems would provide information needed for design.

Lateral Force Resisting System

The risk of damage resulting from lateral drift could be addressed by retrofitting with a concrete shear wall system. This could consist of replacing existing exterior wall bays with new concrete shear walls and/or adding external flying buttresses. Replacing the brittle exterior wall system, consisting of concrete masonry blocks and a brick veneer, with a more flexible system such as metal studs with metal panels should be considered. Additionally, essential MEP systems should be evaluated for support and anchorage; attachments relying on the thin masonry elements should be replaced with attachments to more secure elements.

A rough order of magnitude estimate of probable construction cost for the CUP mitigations is $16 million. The basis of the cost estimate can be found in the structural report in Appendix 3.

Utility Tunnel

The potential for settlement of the old corrugated steel pipe (CSP) utility tunnel between the pile-supported CUP and the pile-supported tunnels under the HQ/P2 buildings could be mitigated by replacing the tunnel with a pile-supported tunnel. It could be feasible to construct a new tunnel around the existing tunnel and transfer support of the utilities to the new tunnel as the existing tunnel is demolished and removed. A second possibility, depending on the condition of the CSP, could be to construct a pile-supported frame underneath the existing tunnel.

3.3.2 Concourse C – Building Sections C1, C2, C3 Mitigation

Liquefaction

While the building structure is supported on piles that will prevent significant settlement, the slab-on-grade and the utility tunnel rely on soil support. The soils could settle a foot or more in a large earthquake. The settlement risk could be mitigated by installing micropiles under the slab-on-grade and alongside the tunnel. Micropiles under the slab would be spaced in general to achieve an effective span of approximately 10 feet, and also located under equipment installations such as baggage make-up devices. Alternatively for the slab-on-grade, a bonded-reinforced concrete topping slab approximately 4 inches thick could be placed over the slab to reduce the number of micropiles required. This would be most feasible in large open areas, such as bag make-up areas. With either approach, voids below the slab will likely be created by liquefaction and soil settlement. The voids would need to be filled with a stable material, such as a low-density pumpable grout, before the slab could be loaded heavily.

Lateral Force Resisting System

The existing moment-frame lateral force resisting system in Concourse C would need to be both strengthened and stiffened to achieve a performance level of Immediate Occupancy, if reliance was to be solely on the moment-frame system. Reinforcing the existing moment frames would be intrusive and disruptive. An alternative to reinforcing the existing structure would be to install a force damping system. This would likely be more economical and less disruptive. A system of fluid viscous dampers installed in diagonally-braced bays would absorb a significant amount of seismic force and help to keep displacements within acceptable limits. There is considerable latitude in the placement of dampers, which would be beneficial for minimizing disruption of functions and spaces. Additionally, the systems function by absorbing the greatest forces at peak velocity of the building, which occurs at zero displacement. This is out of phase with the demand on moment frames for which forces are greatest at maximum displacement, or zero velocity. Thus, a damping system would greatly reduce, or possibly eliminate, the need to reinforce columns and foundations. A detailed analysis would be required to determine the number and placement of dampers.

A rough order of magnitude estimate of construction cost to install micropiles and a 4-inch bonded topping slab for the Concourse C slab-on-grade, micropiles for the utility tunnel, and a viscous damping system in the building is $81 million.

3.3.3 Terminal Ticket Lobby – T1 Mitigation

Liquefaction

Settlement of the building columns and walls of the Terminal Ticket Lobby resulting from settlement of the older, shorter piles in the foundation system could be mitigated by installing new micropiles at each column. This would entail removing portions of the slab-on-grade and penetrating existing pile caps. Settlement of the slab-on-grade could be mitigated by installing micropiles, or by removing and replacing the slab with a structural slab spanning to the reinforced pile caps. A combination of these methods would be possible, potentially minimizing impacts on existing building elements or operations. The existing entry/exit vestibules are soil-supported. Micropiles should be installed under the vestibules to ensure the exit paths remain usable.

Lateral Force Resisting System

The concrete shear walls which form the primary lateral force resisting system can be expected to perform adequately for Immediate Occupancy for the 475-year return period earthquake. Concentric steel braced frames above the mezzanine which brace the mezzanine to the roof and the roof to the shear walls below could be replaced with more ductile buckling-restrained braces for better ductility performance and to limit the forces transferred to columns. Reinforcing of certain structural connections would also be required. Additional piles to resist seismic overturning forces may be needed.

A rough order of magnitude estimate of probable construction cost for Terminal Ticket Lobby mitigations is $47 million.

As part of next steps to define the optimal mitigation scheme for the Terminal Ticket Lobby, the roadway canopy should be reviewed for the anticipated seismic loads and lateral displacements of the parking garage and terminal buildings, as well as internal deformations that could affect the glazing on the canopy. As the P1 Parking Garage supports the east side of the canopy, evaluation of that structure for a compatible performance objective should be included.

3.3.4 Terminal South Node – T2 Mitigation

Liquefaction

Similar to Concourse C, the pile foundations are sufficiently deep that settlement of the building would be minimal. However, the slab-on-grade and the utility tunnel are soil-supported and could settle as much as a foot in a large return-period earthquake. As described for Concourse C, the settlement risk could be mitigated by installing micropiles under the slab-on-grade and alongside the utility tunnel. Spacing of the micropiles under the slab-on-grade would be such to achieve an effective span of approximately 10 feet. Alternatively for the slab-on-grade, a 4-inch thick bonded-reinforced concrete topping slab could be placed over the slab to reduce the number of micropiles required. This would be most feasible in large open areas, such as baggage make-up. With either approach, voids below the slab would likely be created by liquefaction and soil settlement. The voids would need to be filled with a stable material, such as a low-density pumpable grout, before the slab could be loaded. A third alternative for the slab-on-grade would be to replace the slab with a thicker, structural slab.

Lateral Force Resisting System

The shear wall lateral force resisting system in the Terminal South Node has the capacity to meet Immediate Occupancy criteria for the 475-year return period seismic event. However, various structural connections likely would need to be reinforced to improve strength and/or ductility. Additionally, new piles could be necessary to resist seismic overturning forces.

A rough order of magnitude estimate of probable construction cost for mitigations in the Terminal South Node is $36 million.

3.3.5 Terminal Oregon Marketplace South – T3 Mitigation

Liquefaction

As described for the Terminal Ticket Lobby, settlement of the building columns and structural walls of the Terminal OMP South caused by settlement of the older, shorter piles in the foundation system could be mitigated by installing new micropiles at each column. This would entail removing portions of the slab-on-grade and penetrating existing pile caps. Settlement of the slab-on-grade could be mitigated by installing micropiles, or by removing and replacing the slab with a structural slab spanning to pile caps at the building columns. A combination of these methods would be possible.

Lateral Force Resisting System

The lateral force resisting system is composed of a combination of concrete shear walls and steel concentric braced frames. The concrete shear walls can be expected to perform adequately for Immediate Occupancy for the 475-year return period earthquake. However, ductility of the braced frames does not meet current code requirements. The performance of the building would be improved by replacing the braced frames with more ductile buckling-restrained braces and/or reconfiguring the braced frames. Two braced frames should be replaced with concrete shear walls to achieve more uniform performance within the lateral force resisting system. Various structural connections would likely need to be reinforced, and additional piles may be needed to provide resistance to seismic overturning that meets current criteria for Immediate Occupancy.

A rough order of magnitude estimate of probable construction cost for mitigations in the Terminal OMP South is $20 million.

4. PERFORMANCE ASSESSMENTS OF MARINE ASSETS

The following narrative summarizes the performance assessments of the selected key Port of Portland marine assets conducted in this study. The narrative includes a brief description of each asset, summarizes the results of preliminary performance assessments, and outlines potential measures to mitigate the risk of earthquake damage. The potential risk mitigation measures were incorporated into the benefit-cost analyses. More detailed discussion of the assessment of marine facilities can be found in the full report prepared by BergerABAM contained in Appendix 4.

4.1 Background on Marine Facilities Performance Assessment

A seismic risk assessment involves evaluating the risk of seismically-induced damage to a structure and components. For marine structures, a typical detailed evaluation may include development of fragility models that help to define the probability of occurrence of a particular structure damage state as a function of the seismic hazard. In lieu of a detailed quantitative assessment, this study conducted a screening-level qualitative and streamlined quantitative assessment as an initial effort to understand seismic vulnerability and to identify and prioritize potential mitigation projects. The study produced estimates of the damage, repair downtimes, and associated repair costs for several seismic levels for the specific assets that were considered. Based on the damage evaluation, potential mitigation measures to reduce the impact of each seismic event were developed for each asset where appropriate. The mitigation measures with associated estimated costs were incorporated into the benefit-cost analyses to evaluate the financial viability of potential seismic improvements. It is anticipated that detailed engineering assessments will be developed in future Port projects for evaluating specific proposed mitigation measures.

The current industry approach for seismic analysis and design of port marine structures is based on performance under various, specific seismic hazard levels. Different performance objectives are targeted for each seismic hazard level, with life-safety design at the highest seismic level and targeted performance criteria at lower seismic levels. Using ASCE *Seismic Design of Piers and Wharves*[3] code as guidance, a three-level seismic hazard and performance evaluation was established for the assessments of this study. For new designs of structures that are considered essential to the regional economy or post-event recovery (corresponding to a "High" design classification), the three levels including description of the ground motion probability of exceedance (PE) and the performance level, as introduced in Section 2.5 of this report, are:

- Operating Level Earthquake (OLE): 72-year return period (50% PE in 50 years); Performance Target: minimal damage with near-elastic structural response with little or no residual deformation, little or no loss of serviceability of the structure, and no loss of containment of materials.

[3] American Society of Civil Engineers, *Seismic Design of Piers and Wharves* (ASCE 61-14), 2014

- Contingency Level Earthquake (CLE): 475-year return period (10% PE in 50 years); Performance Target: controlled and repairable damage with response in a ductile manner, limited inelastic deformations with repairable damage, loss of serviceability for no more than several months, and no loss of containment of materials.
- Design Earthquake (DE): For the purposes of this evaluation, a 975-year return period (5% PE in 50 years) was used as the upper bound earthquake. For design, the DE is obtained by taking two-thirds of the Maximum Considered Earthquake (MCE), which is a probabilistic earthquake with ground motions at a 2475-year return period (2% PE in 50 years). Performance Target: life-safety protection at the design earthquake per ASCE 7-05.

A general review of original structural drawings was conducted for each facility to understand the seismic force resisting system and to evaluate potential structural deficiencies that could lead to structural damage during a seismic event. In addition to the drawing review, original and retrofit seismic calculations, models, and reports for the waterfront structures were reviewed when available from the Port's record library.

A screening-level vulnerability assessment was made at each of the three hazard levels to estimate expected damage and repair downtime and costs. The assessments were based on the review of available data and considered the current condition of each structure. The high-level qualitative assessments were made using engineering judgment, supplemented with streamlined quantitative assessments using simple structural models where appropriate.

Mitigation measures were identified to improve the seismic performance of the existing marine structures to meet the Contingency Level Earthquake, 475-year return period ground motions. Several alternative mitigation measures may be appropriate at a given facility; however, generally, a single mitigation measure was evaluated for the purposes of this assessment. In cases where seismic upgrades to an existing structure are deemed not feasible or economically reasonable, the appropriate mitigation may be replacement of the facility. Where assessments indicated that damage would be significant and not repairable, estimates of downtime and costs are for demolition and reconstruction.

Estimated downtime and repair/reconstruction and mitigation costs assume only a nominal time and cost for planning and permitting. In normal circumstances, planning and permitting time may vary significantly, anywhere from several months to five years or more. These estimates also do not consider scarcity of resources and demand for construction labor that could be likely after a major seismic event, and which could both extend durations and increase costs.

The seismic risk assessment conducted in the study focused specifically on the berth structure of each asset and did not consider Port-owned or tenant-owned equipment (cranes, conveyors, etc.) and systems that may be supported by the structure. A future, broader system assessment would need to address these elements and may also include other elements, the failure of which could jeopardize the use of the facility. Other elements to consider would include utilities, ground slopes, and transportation systems.

4.2 Performance Assessments of Marine Assets

Performance assessments of the marine assets are summarized in the following. The results of the assessments are presented in Table 5 on Page 43.

4.2.1 Terminal 4 – Berth 410, Soda Export Facility

Description

Berth 410 is part of the Kinder Morgan dry bulk soda ash export facility. The berth is a timber pile-supported pier structure constructed in 1962, and was built as an extension of the Berth 411 wharf structure. The pier has a cast-in-place concrete deck working surface supported by treated-timber stringers, timber pile caps, and timber piles. Pile bents are braced using treated-timber diagonal braces.

The overall condition of the pier appears to be fair, with some of structural members exhibiting deterioration and water-staining, including the diagonal bracing and pile caps. Lower connections of diagonal braces to piles are deteriorated in many locations. Water intrusion from the deck appears to be significant. Live loads on the pier have been restricted by the Port due to the condition of the structure. The primary uses of the pier appear to be as an access path to the mooring dolphin at the outboard end of the pier and as a berthing platform.

Seismic Performance Assessment

The lateral force resisting system of the pier consists of the concrete deck, timber stringers, piles, and braces. The concrete deck and timber stingers distribute the deck seismic forces to timber pile caps and then to timber diagonal braces attached to the plumb timber piles. Diagonal braces begin at the pile caps and extend below deck to a horizontal tie beam. Below the diagonal braces, seismic lateral forces are transferred to the mudline through flexure of the timber piles. The pier was originally designed for seismic lateral loads equal to 3.3% of its weight. Current seismic design codes require larger lateral seismic design forces, which can vary from roughly 10% of a structure's weight for small earthquakes (72-year return period ground motions – Operating Level Earthquake), to more than 30% of its weight for larger seismic events (975-year return period ground motions – Design Earthquake).

In general, the pier has a complete lateral load path. However, the structure is vulnerable due to the age, condition, and limited lateral-load resistance capabilities of the structural elements, particularly at the lower connections of the diagonal bracing to the piles. Additionally, potential post-earthquake soil settlement and soil lateral spreading displacements in larger earthquakes would be significant. For the 72-year return period ground motions, soil lateral spread displacements are estimated to be 2 feet. The pier likely has the ability to flex with 2 feet of lateral soil spreading displacements. It is expected that the current function of the dock will not be compromised in the 72-year event and the mooring dolphin should still be accessible. Damage to the pier structure will likely be minor to moderate and should be repairable. Damage could consist of cracks in the concrete deck, broken bracing members and piles, and localized deformations at bracing and pile connections.

It is understood that the new shiploader tower foundation installed in 2013 was designed to current codes. The tower would be expected to perform well at the 72-year event. However, if the new tower foundation is not properly isolated from the seismic effects of the wharf, the foundation may be subjected to much larger forces imposed on it by the adjacent wharf structure.

There is not a seismic joint between Berth 411 and Berth 410. Ground shaking will likely cause the structures to collide into each other repeatedly, resulting in damage. Damage can be expected to include cracking and spalling at the interface, differential separation of the two structures, potentially causing localized collapse of the deck(s), as well as cracking and yielding in various structural elements.

Repairs to the Berth 410 timber pier following the 72-year ground motions would include repair of cracks in the concrete deck, replacing and retrofitting bracing and pile connections, and replacing some number of the timber piles. The pier is not expected to have a significant downtime for repairs since the 1997 mooring dolphin will not require downtime, and access to the dolphin can be anticipated to remain open. The repair cost is estimated to be approximately $1.8 million, at 15% of the cost of a new structure.

At 475-year return period ground motions, the seismic forces and resulting damage to the piles and bracing will be significantly greater than at the 72-year event. Soil lateral spreading is estimated to be 5.5 feet, and the pier does not have the ability to flex and accommodate this extent of soil displacement. Damage can be expected to include broken bracing elements, broken timber piles below the mudline, permanent deformations, and partial collapse of portions of the pier. Seismic forces from the pier could transfer to the mooring dolphin, possibly damaging the dolphin. Differential lateral movements between the pier structure and the conveyor tower foundation could compromise the function of the conveyor tower, depending on actual interface and connection details. The pier will likely not be usable after the 475-year event.

As noted, at an earthquake with ground motions between the 72-year return period and the 475-year return period, , the need to completely reconstruct the pier can be expected as the result of damage caused by extensive soil lateral spreading deformation. A downtime of 26 to 38 months can be expected for reconstruction, not including extra time that might be required for permitting.

4.2.2 Terminal 4 – Berth 411, Soda Export Facility

Description

Berth 411 is also part of the Kinder Morgan dry bulk soda ash export facility, adjacent to Berth 410. Berth 411 is a concrete wharf structure constructed in 1959. The wharf supports a fixed shiploader and various rail tracks, and was originally designed to support a traveling unloading tower. The construction consists of precast concrete piles, cast-in-place concrete pile caps, concrete rail beams, and concrete deck. A timber pile bulkhead is located at the east end of the wharf, near the bottom of the river embankment, and extends south along the water edge. A cantilever sheet pile cut-off wall was installed in 2004 at the waterside face of the wharf to allow for an increased dredge depth.

Based on a visual inspection from a walk-through of the facility and review of a 2013 condition survey, the Berth 411 wharf structure appears to be in fair condition. Minor damage and deterioration are evident on the piles, and on the concrete pile caps and deck which exhibit cracking and spalls.

Seismic Performance Assessment

The lateral force resisting system of Berth 411 consists of the concrete deck and beams distributing seismic loads to the pile caps, which then load the piles which carry the forces to the soil. Similar to Berth 410, the Berth 411 wharf was originally designed for seismic lateral loads equal to 3.3% of the structure weight. As noted, current seismic design codes specify seismic lateral design forces ranging from approximately 10% of the structure weight for smaller Operating Level Earthquakes to more than 30% of the weight for the Design Earthquake. The sheet pile cut-off wall was designed for seismic loading from a 475-year return period event.

The Berth 411 structure can be expected to perform adequately during small to moderate earthquakes. Damage from ground motions of the 72-year return period event is likely to be relatively minor and repairable – minor cracks in the concrete deck, cracks at the pile caps, and damage to the shorter concrete piles near the top of the slope. Some damage to the timber piles below the bulkhead can also be expected. The wharf should be sufficiently flexible to accommodate the estimated 0.5-foot lateral spreading of the soil at the 72-year event. If seismic isolation from the wharf structure is adequate, the shiploader can be expected to perform well at this level of earthquake.

Repairs to Berth 411 after a 72-year event can be expected to consist of repairing cracks and spalls in the deck surface, pile caps, and piles. Replacing some of the shorter piles near the top of the slope might also be required. The repair cost is estimated at $5.2 million, 15% of the cost of a new structure, and the repair downtime is estimated at five to eight months.

Damage to the wharf from ground motions of the 475-year return period event would be significantly more extensive. Lateral spreading of the soil could be 3 feet or more, which would impose a large lateral load on the piles potentially displacing the piles. As noted for Berth 410, the lack of a seismic joint between Berth 410 and Berth 411 will likely allow the structures to collide into each other repeatedly, causing damage. Damage can be expected to include cracking and spalling at the interface, differential separation of the two structures, potentially causing localized collapse of the deck, as well as cracking and yielding in various structural elements. Additionally, as at all locations, connections to utilities could be lost as a result of soil displacements. Damage from the 975-year event would be even more extensive.

Similar to Berth 410, the damage caused by an earthquake with ground motions between the 72-year return period and the 475-year return period event can be expected to render Berth 411 unusable; the need for complete reconstruction can be expected. A downtime of 26 to 38 months is estimated for reconstruction, not including extra time that might be required for permitting. Since Berth 410 and Berth 411 will both be damaged to the extent that reconstruction is necessary to restore function, it is expected that replacing the two berths with a combined facility would be the preferred approach. An order of magnitude estimate of cost to construct a new combined replacement for Berths 410 and 411 is $42 million.

4.2.3 Terminal 5 – Berth 501, Grain Export Facility

Description

Berth 501 was constructed in 1974 and serves the grain terminal. The facility is a hybrid pier structure consisting of three freestanding, 56-foot-diameter sheet pile cell structures spaced 150 feet apart. The cell structures are backfilled with compacted gravel and are topped with concrete cap slabs providing a working dock surface. Each of the cell structures supports towers and grain conveying equipment. This equipment is founded on piles driven within each cell structure. The cells are interconnected by pile-supported concrete aprons which extend upstream and downstream from the east and west cells, respectively, connecting with a pair of mooring dolphins. The dock structure is connected to shore at the upstream end by a trestle and a retractable sliding bridge section. The trestle is constructed of pipe piles and steel members. The retractable bridge is also constructed of steel. Shoreward of the center cell structure are pile-supported concrete caps that support the grain conveying equipment.

Seismic Performance Assessment

The seismic design criteria used in the original design of the dock structure were not available. However, it can be assumed that lateral seismic design forces – if considered – would have been substantially less than the design forces required by current codes. The seismic lateral force-resisting system of the dock is largely composed of the three separate sheet pile cell structures, where the sheet pile and contained gravel backfill behave as a gravity-based structure. The apron longitudinal seismic lateral forces are transferred to the cell structures and transverse seismic forces are transferred to the piles supporting the aprons. The apron piles transfer seismic forces down to the mudline.

The shiploader towers at each cell structure are individually supported by concrete pile caps and 16-inch octagonal concrete piles driven inside the sheet pile cells. Differential displacement between adjacent cell structures is expected to be small; however, due to the large mass and stiffness of each cell structure, even small differential displacement may impose very large seismic forces into the aprons, depending on connectivity to the cells. The aprons may not have been designed to transfer these forces. The shiploader towers consist of multilevel steel-braced frames with a truss moment frame at the lowest level. The truss moment frame and connections to the dock would be vulnerable to localized yielding.

Walls of the cell structures may be vulnerable to lateral loads from soil spreading, which is expected to be significant at Berth 501. These lateral loads would impose forces that could potentially lead to localized bending or buckling of the sheet pile walls. The dolphin supporting the conveyor tower and the conveyers for the shiploading towers is highly susceptible to lateral spreading displacements, and could undergo large, permanent deformations after a small to moderate earthquake. Conveyor bridges span from a mid-way support tower to the shiploader towers, and any deformation of the conveyor tower and bridges could result in the transfer of large seismic forces.

Ground motions of the 72-year Operating Level Earthquake can be expected to cause significant damage with extensive repairs required. Soil lateral spreading displacements are estimated to be greater than 2.5 feet. Displacements of this extent could fracture and fail the timber piles supporting the aprons. The cell structures may remain stable and resist the soil lateral spread forces at this level of earthquake; however, some localized deformation of the sheet pile walls may occur. The isolated conveyor tower dolphins may displace up to 2.5 feet due to the soil lateral spreading, which would likely damage the conveyor towers, conveyor bridges, and shiploader towers and possibly collapse parts of the conveyor systems.

Downtime for repair of Berth 501 damage from the 72-year return period event is estimated to be 12 to 16 months. The order of magnitude estimate of repair costs is $8.5 million.

At 475-year ground motions, the dock will exhibit significant damage from seismic inertial forces and the estimated 7 to 8 feet of soil lateral spreading displacement. The cell structures, aprons, and trestle will be exposed to large lateral forces from the slope movement. Permanent deformations and settlement of the dock, cell structures, and towers could make repairs impractical. The replacement downtime for a new dock is expected to be 22 to 34 months with an associated order of magnitude cost estimate of $28 million. This cost does not include replacement costs of the conveyors, towers, and other mechanical equipment on the dock.

4.2.4 Terminal 5 – Berth 503, Potash Export Facility

<u>Description</u>

Berth 503 serves the mineral bulk terminal. The structure was built in 1992 and is composed of a concrete deck, concrete pile caps and beams, and plumb concrete piles below the deck.

Battered steel pipe piles are located at each fender. The structure arrangement consists of two continuous longitudinal pile caps (parallel to river) that support the rails for the traveling shiploader crane. At three locations, a heavy-duty deck has been provided across the full width for locating maintenance equipment to service the shiploader and conveyor system. Integral with the deck structure are six fender panels. A trestle that connects the pier to shore exists at the downstream end of the pier.

The overall condition of the pier appears to be satisfactory. Several concrete structural members exhibited cracking and spalling with some rust staining; these conditions are considered typical for a structure of this age.

<u>Seismic Performance Assessment</u>

Seismic loading criteria used in the original design were not available for the dock structure. The lateral force-resisting system consists of a concrete deck that distributes deck forces to supporting concrete pile caps below the deck. The pile caps and supporting piles provide lateral stability of the structure and allow the seismic forces to be transferred to the mudline through flexural stiffness of the concrete piles and axial stiffness of the steel batter piles.

At the 72-year return period ground motions, the dock structure is expected to perform adequately. Lateral spreading is triggered at the 72-year event with estimated soil displacements of approximately 1 foot. The piles can be expected to resist the lateral spreading displacements and forces. Minor to moderate repairs could be required for the deck, piles, and pile caps. Closer to shore, the approach trestle may be significantly damaged by the effects of lateral spreading.

The downtime estimated to repair Berth 503 following a 72-year event is estimated at 5 to 8 months. An order of magnitude estimate of repair cost is $9 million. The repair cost is estimated as 30 % of the cost of a new structure.

A structural capacity assessment of the dock at 475-year return period ground motions was conducted in 2012 by the Port. That assessment concluded that force/capacity ratios of structural elements were near, or just above, code-prescribed capacity for the 475-year event. The flexural capacity of the longitudinal concrete beams was shown to be exceeded by 25%. The assessment indicated that the dock structure would be “damaged but repairable” in a 475-year event. A separate assessment in 2014 concluded that no lateral load-resisting elements would experience stresses more than 10% above the original design values in a 475-year return period event. However, neither of these assessments considered the effects of soil liquefaction and lateral spreading. Similar to other marine locations, soil liquefaction and lateral spreading displacements are expected to be significant at Berth 503 in the event of a major earthquake; lateral displacements at this location are estimated at 7 feet in a 475-year event. Soil displacements of this magnitude can be expected to cause significant damage.

With the damage that is expected to result from the soil displacements in a 475-year or greater event, combined with the effects of inertial loading, it is possible that Berth 503 will not be repairable and will need to be replaced. The replacement downtime for a new dock can be expected to be on the order of 26 to 38 months. An order of magnitude estimate of the replacement cost is $38 million.

4.2.5 Terminal 6 – Berth 601, Automobile Facility

Description

Berth 601 serves as a storage and staging facility for imported automobiles. Constructed in 1989, the facility is a floating dock composed of two steel pontoons connected together and with a working deck surface of asphalt concrete paving. The dock is held in place by four breasting dolphins and a series of eight catenary wire rope mooring lines that are anchored to four mooring dolphins. The dock is connected to shore by a steel transfer span and pile-supported trestle. The transfer span ramp is hinged to accommodate water level fluctuations.

On visual inspection, the overall condition of the dock and approach trestle appears to be good.

Seismic Performance Assessment

The lateral force resisting system of the floating dock consists of the breasting dolphins and catenary wire rope mooring lines anchored to the mooring dolphins. The wire ropes and dolphins will act to restrain the pontoons during a seismic event. The approach trestle is supported by a shore-side abutment and a series of pile-supported bents. Transverse seismic forces of the trestle are transferred to the bents and batter piles and longitudinal seismic forces are resisted by the shore-side abutment. The hinged ramp spanning between the trestle and the floating pontoons limits transfer of forces. Design of the trestle structure was based on the 1985 Uniform Building Code, Zone 2 criteria. The design lateral seismic force for the trestle is approximately 11% of its self-weight, which is approximately equal to current code-level design lateral forces for the ground motions of the 72-year return period Operating Level Earthquake. Code requirements for larger events are higher, exceeding 30% in some cases.

Being waterborne, the floating pontoon components are not expected to experience significant damage from seismic events because seismic inertial forces will be highly dampened. However, soil lateral spreading displacements will be significant, estimated at approximately 4 feet in the 72-year return period ground motions and more than 17 feet in the 475-year event.

The trestle and landward dolphins will be significantly damaged at the 72-year event as a result of the large lateral spreading displacements. The replacement downtime for a new trestle and dolphin is expected to be 15 to 21 months. The cost of replacing the structures is estimated to be on the order of $13 million. The estimate assumes the floating pontoons could be salvaged and reused.

4.2.6 Terminal 6 – Berths 604 and 605, Container Terminal

Description

Berths 604 and 605 are sand-filled cellular sheet pile structures constructed in 1974. Together with Berth 603, these berths serve as the Port of Portland's container terminal. In 1994 to 1995, Berths 604 and 605 were structurally modified to accommodate new container cranes. In 2006, a sheet pile wall was installed in front of Berths 604 and 605 to control ship scour. In 2011 and 2012, partial seismic upgrades consisting of jet grouted columns within the main cells and pile arcs and a combination of jet grouted columns and stone columns landward of the main cells were completed on an 800-foot portion of the wharf. The working surface is asphalt concrete paving. At the face of the dock is a combined steel pile/timber pile fender system.

The overall condition of the wharf was considered satisfactory to good in a 2013 assessment conducted by the Port.

Seismic Performance Assessment

The improvements to Berths 604 and 605 undertaken in the mid-1990s were designed for seismic forces associated with ground motions from an earthquake with a 10 percent probability of exceedance in 20 years, representing a return period of approximately 190 years. Based on assessments completed prior to the seismic upgrades undertaken in 2011 and 2012, it was estimated that the wharf in its then-unimproved state could survive seismic events with a return period up to a 50-year level. The partial upgrades completed in 2011 and 2012 improved the 800-foot portion to survive the 200-year event. The repair time for this event was estimated at 4 to 6 months. A benefit/cost study completed in 2012 by the Port assessed the potential benefits of a seismic upgrade to the entire 1,800-foot length of the wharf to meet the 475-year earthquake with repairable damage (GeoEngineers, 2012). Additional assessments of Berths 604 and 605 were not undertaken in this study.

Table 5 on the following page presents a summary of the performance assessments.

Table 5 – Summary of Marine Facility Seismic Performance

		Status quo (As-is) Structure Damage and Downtime				Structure Damage and Downtime with Partial Retrofit						Structure Damage and Downtime for New Replacement Structure			
Berth	Earthquake Return Period	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Partial Retrofit Measure	Cost of Partial Retrofit Measure	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)
410/411	72 Years	410:Minor to moderate repairs; deck repairs, bracing and pile connections; replacement of 10% of piles; 411: deck repairs, pile and pile cap repairs	$7,000,000	6	5 to 8	No feasible partial retrofit alternative to meet 475-year EQ. Reconstruction is the mitigation	N/A	(see columns to right)				Minimal damage (est. 5%)	$1,500,000	0	0 to 1
	475 Years	Total reconstruction	$42,100,000	32	26 to 38							Controlled and repairable damage (est. 30%)	$9,100,000	3	2 to 4
	950 Years	Total reconstruction	$42,100,000	32	26 to 38							Reconstruction	$42,100,000	32	26 to 38
501	72 Years	Moderate to significant repairs to structures (est. 50%)	$8,500,000	14	12 to 16	Significant ground improvements around dock and in cells; pile/pilecap strengthening; strengthen conveyor tower supports with new batter piles; strengthen conveyor system tower and bridge connections.	$19,500,000	Minor damage (est. 10%)	$1,700,000	3	2 to 4	Minimal damage (est. 5%)	$900,000	0	0 to 1
	475 Years	Total reconstruction	$27,700,000	28	22 to 34			Significant, repairable damage (est. 50%)	$8,500,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$5,100,000	3	2 to 4
	950 Years	Total reconstruction	$27,700,000	28	22 to 34			Reconstruction	$24,100,000	28	22 to 34	Reconstruction	$27,700,000	28	22 to 34
503	72 Years	Minor to moderate repairs including deck repairs, pile and pile cap repairs; significant trestle reconstruction	$9,000,000	6	5 to 8	Ground improvement program along shoreline, strengthen piles/connections, strengthen concrete beams	$13,100,000	Minor damage (est. 10%)	$3,000,000	3	2 to 4	Minimal damage (est. 5%)	$1,500,000	0	0 to 1
	475 Years	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Significant, repairable damage (est. 50%)	$15,000,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$9,000,000	3	2 to 4
	950 Years	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Reconstruction	$35,750,000	32	26 to 38	Reconstruction	$37,800,000	32	26 to 38
601	72 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21	Ground improvements around the approach trestle and abutment, strengthen trestle piles/connections at batter pile bents, retrofit dolphins	$4,500,000	Minor damage	$1,100,000	1	0 to 2	Minimal damage	$600,000	0	0 to 1
	475 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Significant, repairable damage	$5,400,000	6	4 to 8	Controlled and repairable damage	$3,300,000	3	2 to 4
	950 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Reconstruction of a new trestle and dolphins	$12,700,000	18	15 to 21	Reconstrcution of a new trestle and dolphins	$13,300,000	18	15 to 21
604/605	72 Years	Minor damage, possible rail reconstruction (est. 10%)	$10,000,000	3	1 to 4	Full seismic upgrades described in GeoEngineers report that targets 500-year EQ for repairable damage	$15,300,000	Minor damage (est. 5%)	$5,000,000	1	0 to 2	Minimal damage (est. 5%)	$5,000,000	0	0 to 1
	475 Years	Reconstruction	$100,000,000	30	28 to 32			Significant, repairable damage (est. 15%)	$15,000,000	3	2 to 5	repairable damage (est 10%)	$10,000,000	3	2 to 4
	950 Years	Reconstruction	$100,000,000	30	28 to 32			Reconstruction	$100,000,000	30	28 to 32	Reconstrcution	$100,000,000	30	28 to 32

4.3 Mitigation Measures Considered for Marine Assets

Measures identified to mitigate seismic risk for the marine assets assessed in the study are summarized in the following. As noted previously, other mitigation actions could also be considered when the Port initiates more detailed planning for mitigation projects.

4.3.1 Terminal 4 – Berths 410 and 411

Mitigation measures that could be considered for Berth 410 include ground improvements and retrofit and replacement of timber members. However, given the age of the facility, the current condition of the timber pier structure, and the extensive cost of the soil improvements that would be needed to minimize lateral spreading hazards to the pier, seismic upgrades to the existing structure for the 475-year Contingency Level Earthquake are not considered economically viable. Similarly, upgrading the wharf structure of Berth 411 and implementing soil strengthening improvements are not likely to be economically viable. The only mitigation action that would be economically viable is to replace the berths with a modern facility, either at the current Berth 410/411 location or at a new location. As noted previously, it is expected that replacing the two facilities with a single combined facility would be the preferred approach. A replacement pier structure could consist of precast concrete deck panels with cast-in-place concrete pile caps and either prestressed concrete or steel pipe piles. The cost of constructing a combined replacement facility is estimated to be on the order of $42 million.

At Berth 411, separate from the wharf structure, the performance of the conveyor tower and shiploader foundations for the 475-year and larger events could be improved by cutting the wharf deck around the foundations to seismically isolate the foundations from the wharf. The tower structure and supporting foundation could be strengthened to resist lateral spreading displacements during a seismic event.

4.3.2 Terminal 5 – Berth 501

Mitigation measures to improve the performance of the Berth 501 structures to survivability at the 475-year return period ground motions would include soil improvements using stone columns or other methods installed on the river embankment, around the approach trestle abutment, and possibly within the cellular structures. Given potential permitting constraints, these soil improvements may only be feasible above the ordinary high water mark. In that case, the dock structure including the cellular structures would remain vulnerable to a slope failure/lateral spreading event. The existing conveyor bridge tower dolphins would need to be strengthened to resist lateral spreading and seismic inertial forces at the 475-year event by adding new piles. Conveyor bridge connections at towers would also need to be strengthened to prevent the bridges from pulling away from the tower. Connections of the shiploader and conveyor towers to the dock structure would be retrofitted to achieve the strength needed to provide appropriate ductile behavior in the tower frame elements. The cost of mitigation measures to achieve survivability of Berth 501 at the 475-year event is estimated to be on the order of $20 million.

4.3.3 Terminal 5 – Berth 503

Mitigation at Berth 503 to achieve survivability with repairable damage in the 475-year event would consist of a ground improvement program along the shoreline and strengthening of piles, pile connections, and concrete beams. Assuming ground improvements and a structural retrofit cost of 30 percent of a new structure, the estimated cost of mitigations at Berth 503 is on the order of $13 million.

4.3.4 Terminal 6 – Berth 601

Mitigation measures for Berth 601 for the 475-year event could include soil improvement using stone columns installed around the approach trestle bents and abutment. Retrofit of the concrete trestle for the inertial loading at the 475-year event may require installation of new piles at each bent. The estimated mitigation cost to retrofit the trestle is on the order of $5 million. Structural mitigation costs (not including ground improvements) for installing additional piling and improving connections can be estimated at approximately 30 percent of the cost of a new structure.

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5. PERFORMANCE ASSESSMENT OF RUNWAY ASSETS

The Seismic Risk Assessment study evaluated the anticipated seismic performance of the south runway, Runway 10R-28L, at PDX, the north runway, Runway 10L-28R, at PDX, and the main runway, Runway 13-31, at the Hillsboro Airport (HIO). The seismic performance of the HIO main runway was evaluated for a single deterministic M9.0 CSZ earthquake scenario, and the PDX runways were evaluated at the five probabilistic hazard levels discussed in Section 2. The following paragraphs provide a brief summary of the subsurface conditions at each of the airfield assets. More detailed information regarding subsurface conditions and the analyses completed is provided in the geotechnical report, contained in Appendix 2. The geotechnical report also provides references to subsurface investigations completed at the asset locations.

5.1 Summary of Existing Geotechnical Conditions

The HIO site is mantled by alluvial soils that are underlain by basalt bedrock at depths on the order of 1,000 ft. The near-surface subsurface conditions in the vicinity of the runway alignment are somewhat variable to a depth of about 35 feet and include relatively stiff silt and interbedded zones of relatively loose, silty and clayey sand. These zones of sand are at risk of seismically-induced strength loss and settlement. Below a depth of about 35 feet, the majority of the profile consists of stiff, low-to moderate-plasticity silt that is not considered susceptible to liquefaction at the hazard levels of interest. Groundwater was assumed to be at a depth of about 10 feet below the ground surface.

The PDX site is underlain by deep alluvial Columbia River flood deposits mantled with a variable thickness of hydraulically-placed dredged sand fill. In contrast to the HIO site, the majority of the soils at PDX are considered moderately to highly susceptible to seismically-induced strength loss and liquefaction. Groundwater was assumed to be at a depth of about 8 feet below the ground surface for the analyses.

5.2 Seismic Performance Under the Analysis Scenarios

The physical condition of a runway under which aircraft landings and takeoffs are allowed is prescribed by standards established by the Federal Aviation Administration (FAA). Currently, there is a single set of standards, as set forth in Federal Aviation Regulations Part 139, *Certification of Airports*, and detailed in supporting FAA documents. The FAA approves runways and other airfield pavements for use based on compliance with the prescribed standards. It is unknown if the FAA would approve commercial use of a runway in a condition that did not fully comply with the standards, even in an emergency situation such as the aftermath of a major earthquake. There are no specific alternate standards for use of a runway that has been disrupted from its original, FAA-approved condition by an earthquake or by other event or force. In the absence of any such alternate standards, requirements prescribed in Part 139 were adopted for this analysis. With regard to pavement condition, three requirements in Part 139 were determined to be applicable to the analysis. The three requirements, taken from 139.305 Paved Areas (a) are:

(1) The pavement edges must not exceed 3 inches difference in elevation between abutting pavement sections and between pavement and abutting areas.

(2) The pavement must have no hole exceeding 3 inches in depth nor any hole the slope of which from any point in the hole to the nearest point at the lip of the hole is 45 degrees or greater, as measured from the pavement surface plane, unless, in either case, the entire area of the hole can be covered by a 5-inch diameter circle.

(3) The pavement must be free of cracks and surface variations that could impair directional control of air carrier aircraft, including any pavement crack or surface deterioration that produces loose aggregate or other contaminants.

With these requirements as the basis, the analysis considered settlements of up to 3 inches to be a reasonable performance criterion. Differential settlement caused by soil liquefaction is typically assumed to be on the order of one-half of the total magnitude of liquefaction-induced settlement. Following this assumption, 6 inches of total liquefaction-induced settlement at the runway sites was presumed to be the limit that would enable runway use to be continued.

The results of the screening-level analyses indicate there is a low risk that the HIO Runway 13-31 will exceed the allowable deformation criteria outlined above in response to the M9.0 CSZ earthquake scenario. While some settlement is expected, it is anticipated that the runway would be functional, at least for emergency response purposes. It should be assumed that portions of the runway will need to be rebuilt for full resumption of long-term commercial aviation use.

Analyses completed for the PDX runways show a much higher risk of liquefaction-induced settlement and associated differential settlement, resulting in significantly more pavement damage than at HIO. Based on the results of the preliminary analyses for the PDX runways in consideration of FAA standards for pavement condition, it is anticipated that unacceptable differential settlements will occur at seismic events with return periods greater than approximately 225 years if the liquefaction hazard is not mitigated. It is assumed that the 3 inches of vertical offset specified by the FAA may be non-conservative except for emergency response, military-type transport aircraft.

5.3 Estimated Downtime and Repair Costs

The minimum time to fully reconstruct one of the PDX runways post-earthquake is estimated to be approximately 10 months. Project costs to replace the south runway are estimated to be on the order of $77 million, and costs to replace the north runway are estimated to be on the order of $62 million. To estimate the length of time required for smaller post-earthquake repair at each of the selected earthquake return periods for the unmitigated scenario, the 10-month duration assumed for complete reconstruction was pro-rated for a percentage of a runway assumed to be damaged by ground failure due primarily to post-seismic settlement at each return period. Table 6 summarizes damage projections at the various earthquake levels. Figure 5 illustrates estimated runway downtimes.

Table 6 – Runway Performance Summary

		Status quo (As-is) Damage and Downtime				Damage and Downtime with Option #1				
Runway	Earthquake Return Period	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Partial Retrofit Measure	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)
North	72 Years	minimal damage (< 1% of runway)	$250,000	0.50	0.25 to 0.75	Ground treatment with Stone Columns to depth of 40 ft.	negligable damage (assume 0% of runway)	$ -	0	0
	225 Years	moderate damage (≈ 10% of runway)	$6,300,000	2.0	0.75 to 3.0		minimal damage (< 1% of runway)	$ 100,000	0.5	0.25 to 0.75
	475 Years	Significant repairable damage (≈ 25% of runway)	$15,700,000	6.0	4.0 to 8.0		minimal damage (< 1% of runway)	$ 250,000	0.5	0.25 to 0.75
	950 Years	Reconstruction (≈ 50% of runway damaged)	$31,400,000	8.0	6 to 10		moderate damage (≈ 4% of runway)	$ 2,500,000	1.0	0.5 to 1.5
	2475 Years	Reconstruction (≥ 70% of runway damaged)	$62,000,000	10.0	6 to 12		moderate damage (≈ 10% of runway)	$ 6,200,000	2.0	0.75 to 3.0
South	72 Years	minimal damage (< 1% of runway)	$300,000	0.75	0.5 to 1.0	Ground treatment with Stone Columns to depth of 40 ft.	negligable damage (assume 0% of runway)	$ -	0	0
	225 Years	moderate damage (≈ 10% of runway)	$7,700,000	3.0	2.0 to 4.0		minimal damage (< 1% of runway)	$ 100,000	0.5	0.5 to 0.75
	475 Years	Significant repairable damage (≈ 25% of runway)	$19,300,000	7.0	6.0 to 8.0		minimal damage (< 1% of runway)	$ 300,000	0.75	0.5 to 1.0
	950 Years	Reconstruction (≈ 50% of runway damaged)	$38,500,000	10.0	6 to 12		moderate damage (≈ 4% of runway)	$ 3,100,000	1.5	1.0 to 2.0
	2475 Years	Reconstruction (≥ 70% of runway damaged)	$77,000,000	10.0	6 to 12		moderate damage (≈ 10% of runway)	$ 7,700,000	3.0	2.0 to 4.0

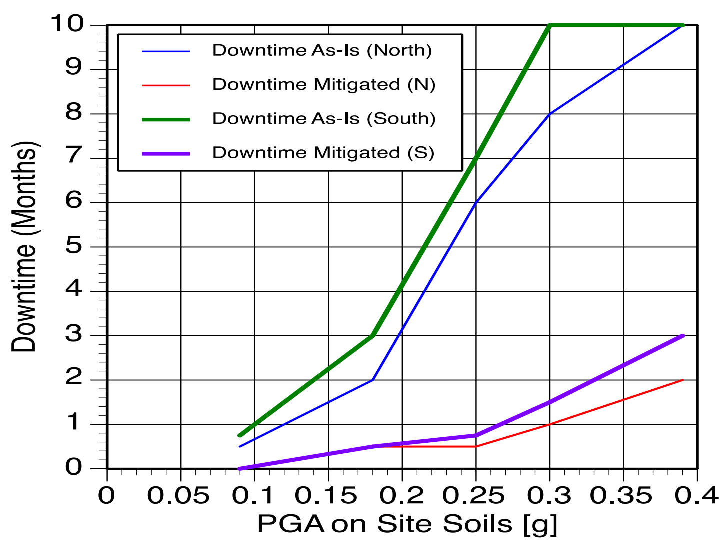

Figure 5 – Estimated Runway Downtimes

5.4 Potential Mitigation Strategies

Three primary mitigation strategies were evaluated for the PDX runways. Although the north runway was not specifically analyzed, it is assumed that soil behavior at the site of the north runway would be similar to the soil behavior at the south runway location with the exception of an increased risk of lateral spreading at the north runway location at higher hazard levels. The three mitigation strategies consist of:

1) Jet Grouting of the portland cement concrete South Runway

   The strategy assumes multiple extended closures to undertake the mitigation without a complete repaving of the runway. Grouting would be through hoes cored in the runway pavement.

2) Stone Column Mitigation of the portland cement concrete South Runway

   The strategy assumes mitigation would be accomplished during a future repaving of the runway.

3) Stone Column Mitigation of the asphalt concrete North Runway
   The strategy assumes mitigation would be accomplished during a future repaving of the runway.

Each of these strategies has been proven to be effective in limiting liquefaction-induced ground settlements. An example of the effectiveness of grouting to mitigate seismic risk can be seen at the Sendai, Japan airport. A grouting program was undertaken for the airport's primary runway in 2008. The grouting program involved a combination of jet grouting and chemical grouting through holes cored in the runway pavement. Jet grouting was used in areas of particular susceptibility to settlement, and chemical grouting was used elsewhere along the runway. The mitigated ground supporting the runway performed well in the 2011 Magnitude 9.0 Tohuku earthquake, with no settlement or ground deformation affecting the runway pavement.

The ground improvement schemes considered for the PDX runways were targeted at mitigation at the 975-year hazard level with acceptance of additional risk at greater hazard levels. The results of the concept analyses indicate that jet grouting to depths of about 30 feet below the pavement surface and stone columns to depths of about 40 feet would likely limit liquefaction-induced ground surface settlements to tolerable levels.

The cost for jet grouting mitigation for the south runway is estimated to be on the order of $137 million. The cost for stone column ground improvements for the south runway is estimated to be on the order of $67 million, and for the north runway is estimated to be on the order of $68 million. The estimates were based on the following assumptions:

- An approximate treatment area of 3,080,000 square feet for the south runway
- An approximate treatment area of 3,095,00 square feet for the north runway

- Treatment areas include a minimum 20-foot width around the runway pavement, and a 50-foot width along the north side of the north runway to limit the risk of liquefaction-induced lateral spreading deformations considering the soils in that particular area
- Stone column treatment to a depth of about 40 feet using a center-to-center column spacing of 8 feet
- Jet grouting treatment to a depth of about 30 feet using an area replacement ratio of approximately 20%

These scenarios and costs were developed for use in the benefit-cost analyses to evaluate the economic feasibility of potential long-term risk management strategies. It is recognized that risk mitigation using ground treatment under an entire runway and surrounding area represents a significant expense that must ultimately be balanced against other viable methods of risk management.

It should be noted that the mitigations considered here address only the runway pavement. Other elements and facilities that are associated with a fully-functional PDX runway, such as navigational aids and electrical power for equipment and lights, could also be affected by an earthquake. A particular concern would be settlement of soil supporting key FAA-owned and operated navigational aids for a runway, such as glide slope antennas, localizers, and Precision Approach Path Indicators. Alignment of this equipment with the runway, both horizontally and vertically, is critical. Settlement of equipment would result in misalignment, rendering the equipment unusable. This in turn would reduce the functionality of a runway, even if the runway survived an earthquake with no damage. Accordingly, any project to mitigate seismic risk to a runway should also consider working with the FAA to mitigate risks to critical equipment.

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6. SEISMIC RISK AND BENEFIT-COST ANALYSES

This section summarizes the seismic risk and benefit-cost analyses conducted in the study. The study synthesized the performance assessments of the assets that were evaluated, the estimated downtimes, potential mitigation strategies, and estimated costs to quantify risks and to estimate benefits to costs. The information presented here is a summary of the seismic risk and benefit-cost analysis report prepared by ImageCat, Inc. ImageCat's full report is contained in Appendix 5.

6.1 Background on Seismic Risk and Benefit-Cost Analyses

The facilities modeled in the seismic risk and benefit-cost analyses included the PDX buildings, marine facilities at T4, T5 and T6, and the PDX runways. The software modeling system SeismiCat[4] was used in the analyses of the PDX buildings, with consideration of the business interruption impacts of runway damage. Spreadsheet-based methods were used in the analyses of the marine facilities and runways.

The scope of the analysis included the following:

1) The SeismiCat online seismic risk management system was used to model the PDX buildings with and without seismic retrofits, as described in Section 3 of this report. The outputs included level of building damage, expressed as a fraction of the building value, and the expected downtime for repair of the earthquake damage. The models include damage from shaking and damage from settlements caused by soil liquefaction.

2) The building-by-building modeling information produced by the SeismiCat online system was imported to the SeismiCat Multi-site tool for portfolio risk assessment. Building replacement values and revenue loss rates associated with each building were used to obtain consequences in financial terms (dollars), considering estimated times to rebuild, or downtime. Both Port-only financial impacts and impacts to the region were considered. The regional impacts from the loss of Port functions are much larger than the impacts to the Port alone. Estimated Port and regional impacts are listed in Table 7 on the following page. The total economic impacts for the Port were determined by multiplying the Port's annual revenue, as listed in Table 1, by the estimated time to rebuild. Similarly, the total regional economic impacts were determined by multiplying the annual regional impacts from Table 1 by the estimated time to rebuild.

[4] SeismiCat proprietary software created and maintained by ImageCat, Inc.

Table 7 – Values at Risk, Selected Port of Portland Critical Facilities

Facility	Asset	Priority Rank	Engineering Estimated Total Replacement Cost	Estimated Time to Rebuild (years)	Estimated Total Economic Impact to the Port	Estimated Total Economic Regional Impact
PDX	Central Utility Plant Bldg & Mech Tunnel	1	$63,000,000	3	$242,400,000	$4,680,000,000
	Runways, Taxiways, Ramps & Lighting	2	$110,000,000	3		
	Terminal Concourse C and Pass Structure	3	$543,000,000	3		
	Main Passenger Terminal Building	4	$936,700,000	3		
	ARFF (Fire) Station	5	$15,200,000	3		
	P2 Parking Structure	6b	$196,000,000	3		
	Ground Maintenance Admin & Shops	13	$11,700,000	3		
	Ground Maintenance Facility	14	$5,580,000	3		
	Ground Maintenance Facility	15	$2,850,000	3		
T6	ICTSI - Berths 604 and 605	7	$100,000,000	2	$22,000,000	$240,000,000
	Yard trailer maintenance	16	$2,970,000	3		
	Electric Shop Bldg, SW of Admin Bldg	17	$1,905,000	3		
	AWC - Berth 601	11	$35,000,000	1.5	$3,000,000	$27,000,000
T5	Portland Bulk Terminal - Berth 503	8	$20,000,000	3	$7,500,000	$294,000,000
	Columbia Grain Facility - Berth 501	10	$25,000,000	3	$9,000,000	$540,000,000
T4	Kinder Morgan - Berth 410-411	9	$40,000,000	3	$10,200,000	$270,000,000
HIO	Runway 12/30	12	$66,000,000	1.5	$4,650,000	$99,000,000
Total			$2,174,905,000		$300,000,000	$6,200,000,000
PORT	HQP2 - Port Administrative Offices	6a	$109,600,000		$300,000,000	$6,200,000,000

3) The SeismiCat multi-site software analyzes losses for a large inventory of earthquake simulations. For this study, that included local earthquakes from sources such as the Portland Hills Fault, as well as large events on the more distant Cascadia Subduction Zone. Further information on the seismic risk analysis methods is contained in Appendix 5. For this study, the SeismiCat Multi-site tool was modified as follows:

   a. Site-specific geologic conditions and ground motion amplification at PDX developed for this study were incorporated. These are depicted in Figure 6, below. "PGA" refers to peak ground acceleration.

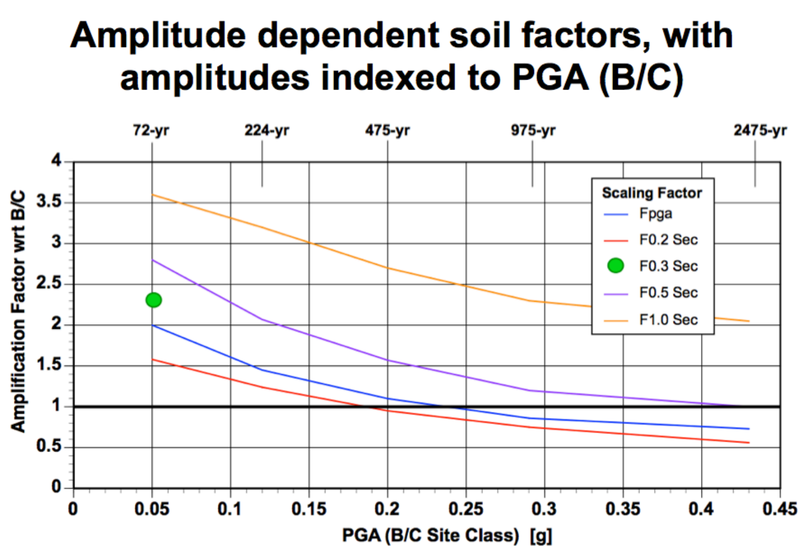

Figure 6 – Site-Specific Ground Motion Scaling Model for PDX

b. PDX-custom logic was implemented for analysis of business interruption losses for PDX facilities (buildings and runways). The custom logic was developed in coordination with the Port, and is illustrated in Figure 7 below.

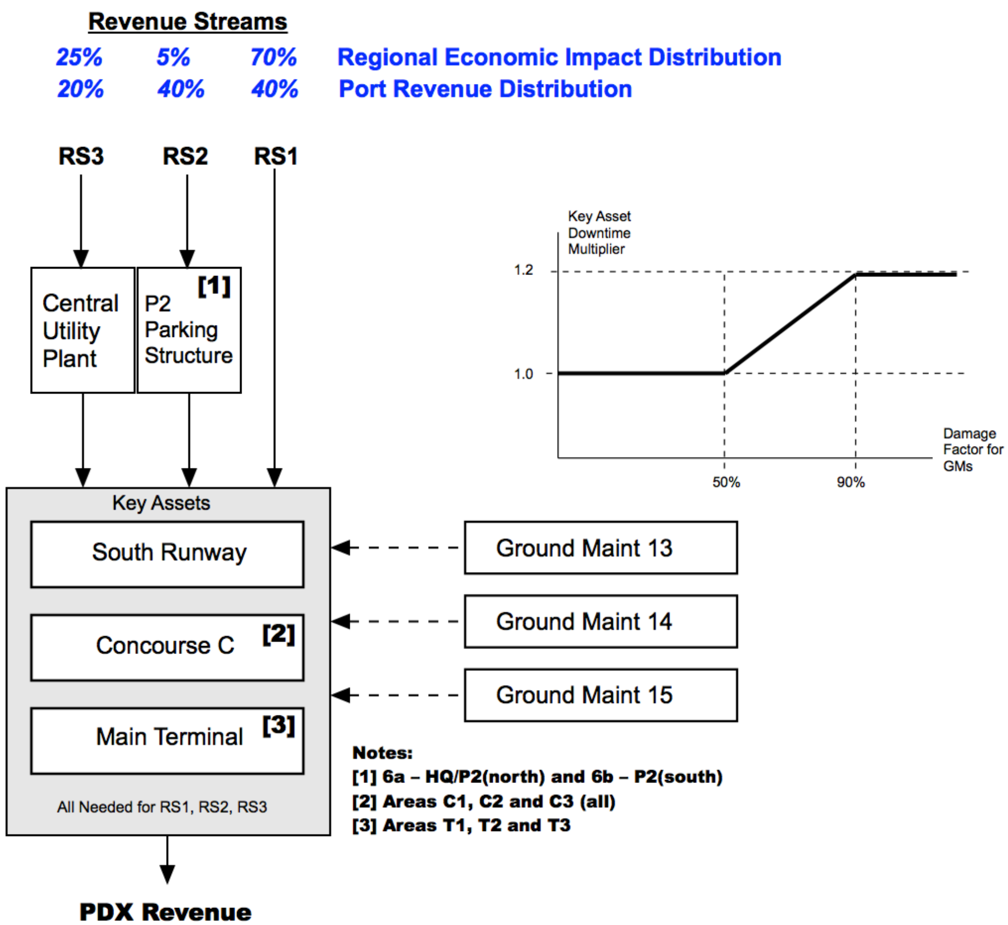

Figure 7 – "Systems" Model for PDX Business Interruption

4) Risks to marine facilities before and after seismic retrofit and/or replacement were evaluated, and the risk analysis was adapted for simplified benefit/cost analysis.

5) Risk and benefit/cost analyses were conducted for eight different cases, consisting of:
   i. Port Only Impacts – Buildings, Existing ("As-is")
   j. Port Only Impacts – Buildings, Runways and Marine Facilities, Existing ("As-is")
   k. Port Only Impacts – Buildings with Mitigation
   l. Port Only Impacts – Buildings, Runways and Marine Facilities with Mitigation
   m. Port and Region Impacts – Buildings, Existing ("As-is")
   n. Port and Region Impacts – Buildings, Runways and Marine Facilities, Existing ("As-is")
   o. Port and Region Impacts – Buildings with Mitigation
   p. Port and Region Impacts – Buildings, Runways and Marine Facilities with Mitigation

Existing/Status Quo ("As-Is") Risks

Figure 8 below shows the results from the seismic risk analysis of the selected critical facilities, under status quo conditions (i.e., prior to any retrofit). The solid lines represent risks to the PDX facilities, including downtime induced by damage to runways. The dashed lines include approximate impacts to the marine facilities. The cases represented in the figure are a, b, e, and f, as introduced previously. The post-retrofit cases (cases c, d, g, and h) are presented at the conclusion of this section.

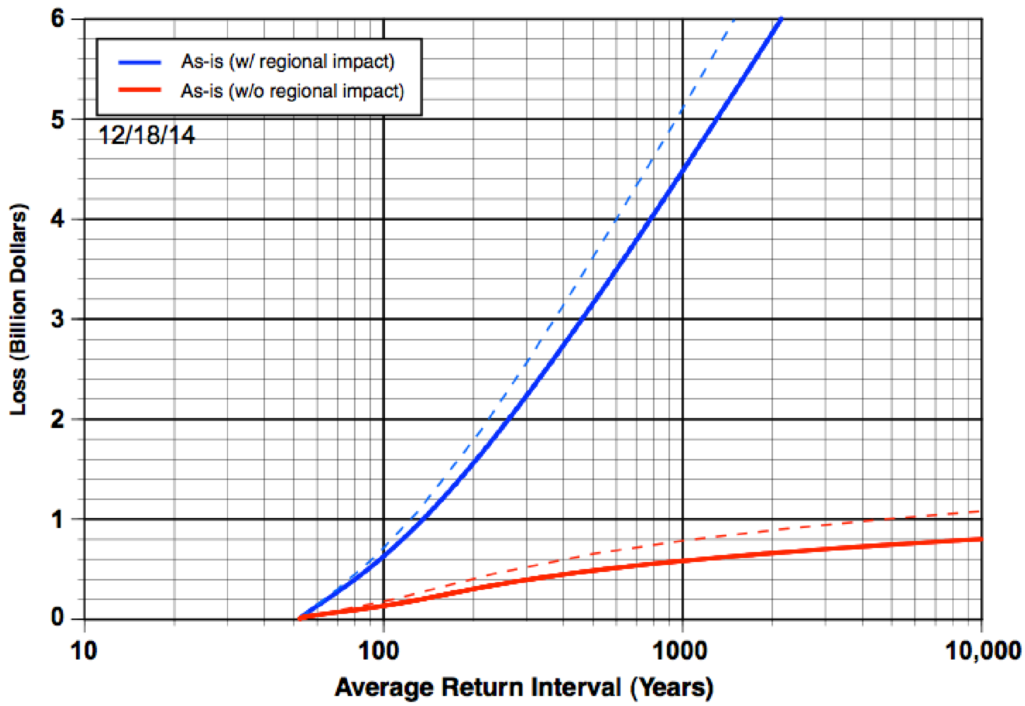

Figure 8 – Seismic Risks to Critical Facilities, As-Is

In considering PDX assets only – the solid lines in Figure 8 – without regional impacts, the 100-year recurrent loss level is on the order of $100M. For increasing return periods and the corresponding increase in earthquake hazard intensity, losses grow, until PDX losses are about $1B for a return period of 2,000 years. When regional economic impacts are considered, the 100-year recurrent loss level exceeds $600M. The losses to PDX and the region approach $6B for a return period of 2,000 years.

The increased losses for the blue curve show the substantial effect of considering the impacts that functionality of PDX has on the region. The dotted lines approximate the additional impact of the marine facilities and runways repair costs, over and above the losses at PDX. In considering actions to take to reduce earthquake losses and improve regional seismic resilience, the blue curves provide strong motivation for mitigation actions as a result of the key role the Port's facilities have in the region's economy, as well as the role the Port's facilities would have in an earthquake recovery effort for the region.

6.2 Seismic Risk Methods and Benefit/Cost Analysis

6.2.1 PDX Buildings

Seismic risks for the PDX buildings were analyzed using the SeismiCat multi-site software, as noted in the background discussion. A large set of earthquake simulations is used in the model to represent the full range of future earthquakes, both in magnitude and in location. The set of simulations derives from the models of the 2008 United States Geological Survey (USGS) National Seismic Hazard Mapping Project [Petersen et al., USGS Open File Report 2008-1128]. Each simulation depicts the geographic distribution of earthquake hazards for the assumed fault rupture and earthquake magnitude. Losses for each of the buildings are estimated for each earthquake simulation. The damage at each building is computed based on the simulated ground shaking and other seismic hazards (e.g. liquefaction), and the vulnerability of the buildings as modeled in the study. For each earthquake, losses are summed for all of the PDX buildings considered, and the downtime losses are found using the "Systems" Model for Business Interruption as introduced in Section 6.1. The losses, and their uncertainty, are then related to the probability of occurrence for the simulation, to allow construction of risk curves and other probabilistic results. As noted, further information on the SeismiCat multi-site software and risk analysis methods is contained in Appendix 5.

6.2.2 Marine Facilities

As the SeismiCat multi-site software does not include marine facilities, the marine facilities were assessed at specific hazard levels corresponding to defined return periods, using a spreadsheet-based method. A similar approach was used for analysis of the PDX runways.

6.2.3 Benefit/Cost Methods

Benefit/cost analysis (BCA) compares the expected benefits of a candidate retrofit alternative to the costs to implement the alternative. As such, BCA requires probabilistic risk analysis. Probabilistic risks are defined by severity of consequences and their annual probability or frequency. Some consequences are easily assigned economic impact – repair costs, for example. Some consequences require economic analysis – for example, the financial impacts of critical facility relocation. Other consequences are difficult or controversial to translate into simple economic terms – the human casualty toll, for example.

The decision framework requires risk analysis for 'status quo' risks for each facility, and risks to each existing facility with the implementation of each retrofit alternative. For calculations of benefit to cost:

– The benefit from each retrofit alternative is found as the reduction in economic (or other) consequence associated with the retrofit alternative with respect to the status quo or as-is state.

- The estimated benefit from a particular retrofit alternative for each simulated earthquake event (or hazard level) is multiplied by the annual frequency of the event (or hazard level) to compute the expected annual benefit (i.e., the annual reduction in cost from earthquake damage with implementation of the retrofit alternative). The total annual benefit is found by a probabilistic summation of the annual benefit for all earthquake events.
- The present value of future benefits from risk reduction afforded by a particular retrofit alternative is found by assuming that the expected annual benefit occurs each year over the remaining life of the building, and treating this as an annual series of payments. Using time-value-of-money, the present value of this series is computed. This present value benefit is divided by the current estimate of the cost of the retrofit alternative under consideration to obtain a benefit-to-cost ratio. One important variable is the effective interest rate or the minimum attractive rate of return used in converting an annual series of payments to its present value.

Decisions regarding seismic retrofit alternatives occur within a stakeholder and facilities management context – specific project criteria, goals, decision alternatives, and decision frameworks. This broader perspective includes questions such as whether to implement a seismic retrofit alternative beyond minimum code requirements or to accept the level of damage expected for a code-minimum baseline case.

6.3 Marine Facilities Benefit/Cost Analysis

Performance evaluations of the marine facilities included in the study are summarized in Section 4. For each berth, analyses were performed for seismic hazards corresponding to several return periods – 72 years, 475 years and 975 years. These correspond roughly to an operating level earthquake, a contingency level earthquake, and a design earthquake, respectively, as described in Section 4. For these various earthquake severities, damage and downtime were predicted, together with expected repair costs. The analyses were then repeated, for the cases in which the marine facilities would undergo partial retrofits or replacement. This formed the basis for estimating the approximate benefit-to-cost ratio for the marine facilities.

For each hazard level, the reductions in loss afforded by the partial retrofits compared to status quo were multiplied by the annual frequency of occurrence of the hazard level, so that an average annual benefit could be computed. This annual benefit was then converted to present value, and divided by the cost of the retrofit option, to obtain a benefit-to-cost ratio. The same procedure was followed for the full replacement options. The results are presented in Table 8 on the following page.

The results show that all of the partial retrofit options appear to be cost-effective with a benefit-cost ratio greater than one, when regional benefits are included. In particular, partial retrofits to Berths 501 and 601 appear to provide good value with benefit-cost ratios around 3. When considering full replacement, again Berths 501 and 601 appear to provide the best return on investment. However, mitigations at Berth 503 and at Berths 604/605 also show good value with benefit-cost ratios of around 2.

Table 8 – Benefit / Cost Results, Marine Facilities

Summary, Benefit / Cost Analysis, Marine Facilities
5% discount rate used. Benefits accrued over 50 years.

		Annual Downtime Cost				Benefit / Cost Ratios			
Berth	Replacement Cost New	Port Only	Region	Partial Retrofit	Retrofit Cost	Port-Only, Partial Retrofit	Port-Only, Replacement	Regional Impact, Partial Retrofit	Regional Impact, Replacement
410/411	$42.1M	$3.4M	$90M	No feasible partial retrofit alternative to meet 475-year EQ Reconstruction is the mitigation	N/A	N/A	0.12	N/A	0.77
501	$27.7M	$3M	$180M	Significant ground improvements around dock and in cells; pile/pilecap strengthening; strengthen conveyor tower supports with new batter piles; strengthen conveyor system tower and bridge connections.	$19.5M	0.36	0.18	3.47	3.17
503	$37.8M	$2.5M	$98M	Ground improvement program along shoreline, strengthen piles/connections, strengthen concrete beams	$13.1M	0.29	0.13	1.77	0.92
601	$13.3M	$2M	$18M	Ground improvements around the approach trestle and abutment, strengthen trestle piles/connections at batter pile bents, retrofit dolphins	$4.5M	1.05	0.38	2.85	1.04
604/605	$100M	$11M	$120M	Full seismic upgrades described in GeoEngineers report that targets 500-year EQ for repairable damage	$15.3M	0.51	0.08	2.24	0.38
	$220.9M				$52.4M				

6.4 PDX Runway Performance Considerations

Section 5 summarizes the results of the seismic evaluation of the PDX north and south runways undertaken in this study. For each runway, analyses were performed for seismic hazards corresponding to several return periods – 72 years, 225 years, 475 years, 950 years and 2,475 years. These levels span the range of earthquake events that were evaluated in the study. For these various earthquake severities, damage and downtime were predicted, together with expected repair costs associated with the predicted damage. The analyses were then repeated for the cases in which mitigation of the seismic risk, as outlined in Section 5, was implemented. This formed the basis for the analysis of approximate benefits-to-cost for the runway mitigations.

Downtimes for the runways are critical, as a functional runway is necessary for aircraft operations. The cost-effectiveness of seismic improvements to the runways must be considered within the context of the operations of the rest of the PDX facilities. To this end, downtime relationships were developed for the runways with and without mitigations for a range of ground motions. The estimated downtimes are illustrated in Figure 5 in Section 5.

Separate benefit-cost analyses were not conducted for the runways. Instead, the costs and benefits for mitigation for the south runway are included in the benefit-cost analysis undertaken for the PDX facilities.

6.5 PDX Facilities Performance

Section 3 presents details of the structural systems and expected earthquake damage for each of the PDX buildings evaluated in the study. Section 3 also identifies potential seismic risk mitigation retrofits for key assets consisting of the Central Utility Plant (CUP), portions of the terminal, and Concourse C. The buildings were modeled in their status quo condition in the SeismiCat online software, and then were modeled again to simulate their performance after the completion of seismic retrofits. The effects of soil liquefaction are particularly important, and short pile foundations and slabs-on-grade will be subject to large liquefaction-induced settlements in high levels of earthquake shaking. For example, the CUP may experience settlements of one foot to 1.5 feet in ground motions with an average recurrence of 500 to 1,000 years. High levels of damage can be expected to the slabs-on-grade.

The rough order of magnitude estimate of cost to retrofit all of the buildings – the CUP, portions of the main terminal, and Concourse C – totals approximately $200 million. The study undertook to evaluate the effectiveness of the building retrofits and identify the most cost-effective. Figure 9 illustrates the relative effectiveness of the potential retrofits expected for each of the buildings, for the level of seismic hazards that recur on average every 500 years.

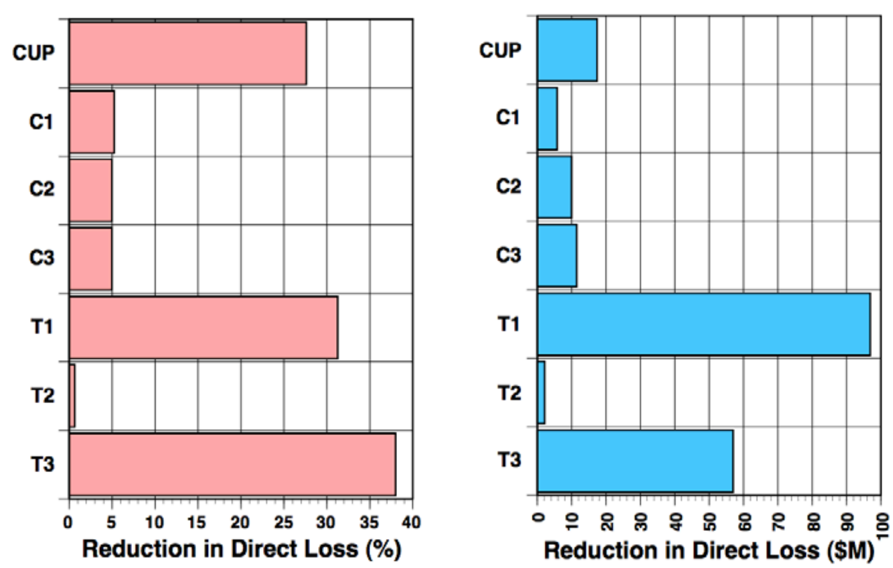

Figure 9 – Comparison of Retrofit Effectiveness, PDX Facilities

From this comparison, it is apparent that the retrofits for the CUP and Terminal units T1 and T3 produce the greatest benefits in loss reduction. Concourse C – sections C1, C2 and C3 – and Terminal unit T2 are expected to perform relatively well in their existing state; thus, the opportunities for cost-effective mitigation for these buildings are not as great.

6.5.1 Seismic Risk Results – PDX Facilities

Figure 10 below presents risk curves for the PDX facilities showing the impacts of seismic retrofit, considering both Port-only economic risks or impacts, and Port-plus-regional economic impacts.

The red and blue curves show the status quo risks, with and without regional economic impacts. The red curve shows the Port-only risks for the status quo condition. For this case, repair costs for earthquake damage are about 2/3 of the values shown by the red curve, with the remainder being Port revenue losses. The blue curve depicts combined economic risks to the Port and the region for the status quo condition. Risks increase substantially when regional impacts are considered.

The green and brown curves illustrate the reduced risks projected with a comprehensive program of seismic retrofits for the PDX facilities considered. The brown curve represents Port-only risks, and the green curve includes regional risks. For the post-retrofit case, repair costs for earthquake damage are about 90 percent of the values shown by the brown curve, with the remainder being fairly small Port revenue losses.

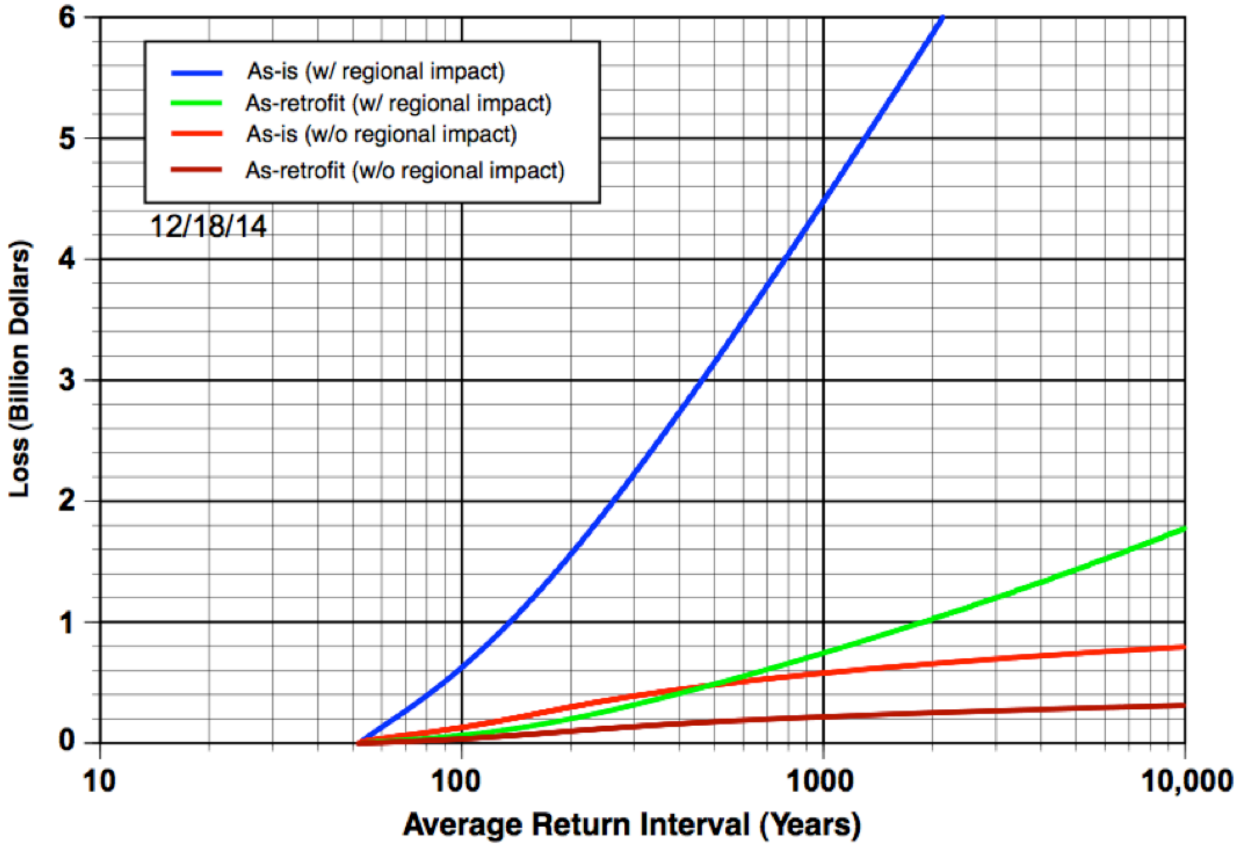

Figure 10 – Building Damage + PDX B.I. (including runway impacts)
B.I. = Business Interruption

6.5.2 Benefit/Cost Analysis Results – PDX Facilities

Table 9 shows the results of benefit/cost analysis for the complete set of the PDX assets for which mitigation retrofits were considered, with and without regional impacts. The assets include the CUP, Concourse C, three units of the main terminal T1, T2, and T3, and the South Runway.

Table 9 – Benefit / Cost Analysis Results, Comprehensive Retrofits of Key PDX Assets

Average Annual Loss (AAL) **Benefit**

All PDX Retrofits + South Runways

As-is		As-retrofit		∂AAL		n	rate	Present Value	
With regional	w/o regional	With regional	w/o regional	With regional	w/o regional			With regional	w/o regional
24,365,730	3,927,906	3,660,681	1,372,155	20,705,049	2,555,751	50	0.05	377,989,831	46,657,600

South Runway Damage Reduction	$ 2,644,571	$ 2,644,571
Total	380,634,402	49,302,171
BCR	**1.4**	**0.18**

CUP	$ 16,000,000
C1	$ 14,000,000
C2	$ 31,000,000
C3	$ 36,000,000
T1	$ 47,000,000
T2	$ 36,000,000
T3	$ 20,000,000
South Runway	$ 67,000,000
	$ 267,000,000

As indicated in the table, the benefit-cost ratio considering Port-only benefits is 0.18. When regional benefits as well as Port benefits are considered, the benefit-cost ratio increases to 1.4. Table 10 shows benefit-cost results considering a smaller group of the PDX building assets – the CUP, and Terminal units T1 and T3 – which showed highest cost effectiveness of mitigation retrofits as illustrated in Figure 9, together with the South Runway. The benefit-cost ratio considering only Port benefits is approximately 0.3, while the ratio considering both Port and regional benefits is 2.2. In addition to the high cost-effectiveness of mitigations for these assets, this smaller group of assets together represents a subset of PDX that would potentially enable continuing function of the airport if the assets were to survive an earthquake intact. The assets would provide for passenger terminal function, security checkpoint function, aircraft passenger enplaning and deplaning function, and aircraft landing and takeoff.

Table 10 – Benefit / Cost Analysis Results, Retrofits of Selected Key PDX Assets

Alternative with Selected Retrofits (CUP, T1, T3 and South Runway)

As-is		As-retrofit		∂AAL		n	rate	Present Value	
With regional	w/o regional	With regional	w/o regional	With regional	w/o regional			With regional	w/o regional
24,365,730	3,927,906	6,175,717	1,716,138	18,190,013	2,211,768	50	0.05	332,075,521	40,377,872

South Runway Damage Reduction	$ 2,644,571	$ 2,644,571
Total	$ 334,720,092	$ 43,022,443
BCR	**2.2**	**0.3**

Selected Mitigation Costs

CUP	$ 16,000,000
T1	$ 47,000,000
T3	$ 20,000,000
South Runway	$ 67,000,000
	$ 150,000,000

Benefit-cost ratios greater than one (1) are generally considered to be indicative of investments for which the payback period would be deemed reasonable. The 1.4 benefit-cost ratio for mitigation of the full set of assets evaluated represents a reasonable payback. The 2.2 benefit-cost ratio for retrofits of the selected key assets represents a strong payback. It should be noted that the benefit-cost ratios calculated for Port-only benefits appear to be relatively low. This is a result of the non-profit nature of the Port's business. The revenues generated for the Port by the Port's facilities represent only a portion of the total revenues generated directly by the facilities; revenues generated by the tenants far exceed the Port's revenues. Thus, consideration of the regional economic benefit as well as the economic benefit to the Port is important.

It should be noted that the economic contributions from life-safety enhancements that would result from the potential mitigations are not considered in the model used in this study. As the terminal and concourse are principal high-occupant areas, life-safety benefits would have some effect increasing benefit-cost ratios for Port-only benefits. FEMA procedures for benefit-cost analysis allow such benefits to be considered, although considerable analysis is necessary to quantify the benefits. The life-safety contribution would be relatively small in comparison to the regional economic benefits, but could be considered in a future study.

The dependency of building functions and operations on lifelines (roads, railroads, waterways) and utilities (power, gas, water, telecomm) was also not considered. With liquefaction and other seismic hazards, damage to lifelines and utilities may impact downtimes and increase regional economic losses.

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7. CONSIDERATIONS FOR CRITICAL NON-PORT ASSETS AND LIFELINE NETWORKS

This Port-wide seismic risk assessment was focused on assets that are owned and maintained by the Port of Portland. The seismic performance of the assets was evaluated using approximate methods of engineering analyses with the goal of incorporating the performance assessment into a comprehensive, portfolio-level benefit-cost analysis. It is acknowledged by the project team and the Port of Portland that continued operations of the Port following the various scenario-level seismic events will be dependent not only on the performance of the on-site assets, but also on the numerous non-Port assets and lifelines serving the Port's facilities. Some of these assets and lifelines include:

- Gas and electric power
- Aviation and maritime fuel
- Water and wastewater utilities
- Telecommunications
- Airline and shipping companies' systems
- Surface transportation systems, including highway, rail, and intermodal links
- Maritime transportation system, including the mouth of the Columbia River jetties and shipping channels
- Columbia River levee system adjacent to PDX

This study has not addressed the seismic performance of off-site lifelines and the interdependencies associated with these various lifelines. Subsequent risk assessment for natural hazards should include these lifelines to the extent possible. This will require Port interaction with the owners and agencies responsible for the specific lifelines. Several current and recent seismic hazard evaluations have been commissioned by organizations overseeing lifelines that serve the Port, for examples Oregon Department of Transportation, Bonneville Power Administration, and City of Portland Water Bureau. The additional effort to update this study with the results of those seismic evaluations in a future phase of work, or otherwise incorporate in a future study, would support the intent of the Oregon Resilience Plan and benefit the Port in future risk management evaluations.

Additional discussion regarding dependency of the Port on regional lifelines and utilities is contained in Appendix 6.

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8. CONCLUSIONS AND RECOMMENDATIONS

The Seismic Risk Assessment Study was undertaken by the Port of Portland to begin to gain an understanding of the vulnerability of the Port's assets to damage from earthquakes and to evaluate the benefit-cost relationship of potential projects to mitigate the vulnerability. The study found that the majority of the Port assets evaluated face some degree of risk from seismic hazards. The degree of risk varies among the assets, based in part on the age and type of construction. As noted in the report, the age of the various structures evaluated ranges from a few years to more than 70 years. Each of the structures was designed and constructed to meet the performance standards of the building code that was in effect at the time that structure was built. Earlier building codes did not recognize the seismic hazards of the region, and thus greater vulnerability is inherent in the older structures. Structures designed to more recent codes and to the current building code typically have greater capacity to resist seismic forces. However, even the newer buildings have risk of damage from a significant earthquake given that the seismic component of building codes is targeted at life-safety and collapse prevention, not preservation of property or function. Additionally, the majority of Port assets are founded on soils that are subject to liquefaction in an earthquake, particularly at PDX and the marine terminals. Soil liquefaction results in settlement and lateral spread, which will damage soil-supported ground floors of buildings as well as marine piers and wharves.

Any appropriate retrofit mitigation action will reduce the risk of damage and improve resilience, although the mitigation may not be cost-effective. The benefit-cost analyses undertaken in this study for potential seismic risk mitigation actions for selected assets show that certain mitigations, or mitigation for certain assets, offer particular cost-effectiveness. Specifically, retrofit mitigation actions for the selected PDX assets evaluated and for marine Terminal 5 – Berth 501 and Terminal 6 – Berth 601 offer good return on investment considering Port and regional economic impacts.

Based on the results of this study, it is recommended that the Port move forward with risk mitigation projects for PDX and at marine terminals. The total estimated cost of the potential mitigation retrofit projects identified for the selected PDX assets evaluated is estimated to be on the order of $270 million. The PDX assets include three units of the passenger terminal - T1 Terminal Ticket Lobby, T2 Terminal South Node, and T3 Terminal Oregon Marketplace South, Concourse C, the Central Utility Plant, and the South Runway. The total cost of potential mitigation retrofits identified for the selected marine facilities evaluated – Berths 501, 503, 601, and 604/605 – is estimated to be on the order of $52 million, not including Berths 410/411 for which the only feasible mitigation is complete replacement with an estimated cost on the order of $42 million.

Recognizing the high cost of these investments, it will be necessary to prioritize mitigation projects. Optimally for PDX, the focus would be on retrofitting the most vulnerable assets in the group of assets previously identified as representing a subset of PDX facilities that would enable continuing function of the airport. Together, the assets consisting of the three units of the passenger terminal, Concourse C, the CUP, and a runway would provide for passenger terminal function, security checkpoint function, aircraft passenger enplaning and deplaning function, and aircraft landing and takeoff. For the marine facilities, the focus should be on the facilities that provide the greatest value to the Port and the region. Accordingly, specific recommendations of this study are as follows:

1. **PDX Runway**

   Mitigation of risks to a PDX runway should be a top priority. Given the liquefaction potential of the ground at PDX, even a small and probable seismic event will cause ground settlement and distortion of pavement to some extent. This would result in a high probability of a repair project that would take the runways out of service for some length of time. Without a usable runway, the airport would not be functional. Further study would determine if the mitigation should be for the South Runway or the North Runway, if the optimal mitigation action would be stone columns or jet grouting, and if the mitigation would be undertaken as a retrofit or as part of a scheduled reconstruction project. If the mitigation was to be executed as part of a scheduled reconstruction project, the North Runway may be the better candidate for mitigation as the asphalt concrete North Runway will need to undergo reconstruction much sooner than the relatively new portland cement concrete South Runway. Planning for a runway mitigation project should include discussions with the FAA about physical condition requirements for a runway to remain in service after an earthquake, and about the potential for improving the survivability of critical FAA-owned navigational aids.

2. **PDX Terminal**

   A terminal mitigation project should be pursued as a second priority. The terminal is necessary for passenger check-in functions and baggage handling. The focus of a mitigation effort should be on terminal units T1 – Ticket Lobby and T3 – Oregon Marketplace South, for which mitigation actions show the greatest cost-effectiveness. Potential mitigation measures would consist of a combination of installing micropiles to strengthen foundations and support slab-on-grade ground floors and exit vestibules, and improving the lateral force resisting systems, as outlined in Section 3 of this report. Further study would determine the optimal mitigation actions. The mitigation would ostensibly be part of the Terminal Core Redevelopment project that the Port has initiated; that project would provide an avenue and mechanism to accomplish the seismic retrofits.

3. **PDX Central Utility Plant or Concourse C**

   A mitigation project for either the CUP or Concourse C should be a third priority. A functioning CUP is critical for full operation of the terminal and airfield functions. Based on the preliminary evaluations of this study, the CUP is vulnerable to settlement induced by soil liquefaction and to lateral drift of the structure. Potential mitigations would include installing micropiles under load-bearing structural elements and critical equipment, and adding a concrete shear wall system. Further study would confirm the vulnerability of the

CUP and determine the optimal retrofits. If the further study finds that the CUP is not as vulnerable as believed, consideration should be given to mitigating the risks at Concourse C as the third priority. Potential mitigation for Concourse C would consist of installing micropiles to prevent settlement of the slab-on-grade ground floor and improving lateral force resistance by adding a force damping system. As an alternative, a future replacement of Concourses A and B could be designed and constructed for the desired performance level, and thus avoiding the disruption of implementing upgrades to Concourse C.

4. **Marine Terminal T6 – Berths 604/605**

   Mitigation at Terminal 6 – Berths 604/605 should be completed, as the top priority for the Port's marine assets. As described in Section 4, an 800-foot portion of the 1,800-foot wharf underwent a ground improvement upgrade with jet grouted columns and stone columns. The viability of this mitigation method has been demonstrated. A project to mitigate the remaining 1,000 feet of the wharf would improve the resilience of the entire facility to withstand a large earthquake. Berths 604/605 would likely be the most important Port marine asset in supporting a regional rebuilding effort in the aftermath of a major disaster.

5. **Marine Terminal T5 – Berth 503**

   Mitigation at Terminal 5 – Berth 503 should be the second priority for the marine assets. Berth 503 operates under the most stable, long-term lease of the Port's marine facilities. Seismic vulnerabilities should be mitigated to keep this facility in business for the long term. Potential mitigation actions would consist of ground improvements such as stone columns along the shoreline and strengthening of piles, pile connections, and concrete beams. Further study would define the optimal mitigation strategy.

For each of these potential projects, next steps will include:

- Detailed geotechnical site assessments to confirm design seismic response spectra, as well as local liquefaction effects on both free-field ground deformations and pile-supported foundations.
- Consideration of higher-level structural analysis, such as non-linear or non-linear time-history computer analyses in order to obtain the most accurate understanding of the likely structural behavior.
- Development of fragility models to more accurately understand the probability of occurrence of a structure damage state as a function of the seismic hazard.
- Assessment of MEP systems critical for function in the Immediate Occupancy condition, including site surveys of existing support and bracing conditions. This may include adjacent non-critical systems that could impact the critical system during a seismic event.

- Architectural review of exterior enclosure systems and drift compatibility, along with interior systems critical for function or potential falling hazards, for the Immediate Occupancy condition.
- Further exploration of potential mitigation measures, and confirmation of the optimal mitigation strategy. Seismic mitigation on existing, operating facilities is inherently disruptive and expensive. For that reason, a comprehensive evaluation of potential measures and alternatives is essential. A relatively high level of effort in the design phase of any seismic strengthening project not only ensures that the desired objectives are achieved in the most efficient manner, but can also lead to significant savings in construction cost and time.
- Refinement of potential projects for more accurate pricing and scheduling, including review in the context of broader Port master planning.

Beyond the specific, prioritized project recommendations, this study offers the following additional recommendations:

- Evaluate the benefit of designing each new project for greater seismic resilience than required by Building Code. Considering that code requirements for seismic design forces are based on life-safety and collapse prevention, not on property preservation or operational continuity, structures designed to code cannot be expected to maintain uninterrupted functionality after a major earthquake. Generally, the cost premium of constructing a building or other structure to a higher seismic performance level will be a relatively small percentage of the basic cost of construction. Based on function and importance of survivability, planning for each new project should include an assessment of the cost and benefit of building the project to resist seismic forces above code requirements. Enhanced structural systems, stronger connections and supports for MEP equipment and systems, and seismic joints for utility interfaces are primary considerations.
- Identify and evaluate mitigations for other key Port assets. This study identified and evaluated potential mitigation actions for only a limited number of the Port's key assets. A similar effort should be undertaken for other assets considered to be critical for the Port's functions. This will address the assets that were assessed in this study but for which mitigation measures were not identified, such as the ARFF facility and the PDX Ground Maintenance Facilities, and may include other assets as well that are deemed to be critical. Soil liquefaction leading to ground settlement is a concern for all PDX and marine facilities; all structures with soil-supported elements are vulnerable. Ground improvements at the ARFF facility, for example, might be considered to prevent damaging settlement. Consideration should be given to the long-term replacement of buildings such as the Ground Maintenance Facilities and the buildings at Terminal 6 that have inherent structural vulnerability.
- Establish a plan for extricating aircraft rescue and firefighting vehicles from the ARFF facility. As noted in the report, the ARFF facility is constructed on a mat foundation. While the building

itself will likely survive a major earthquake, significant settlement can be expected. The settlement could result in the doors of the equipment bays becoming inoperable. While the ARRF facility itself is not essential for the operation of the airport, the availability and readiness of the emergency response equipment is essential. Until such time that a ground improvement project might be undertaken to prevent settlement of the facility, a near-term plan should be established for extricating the equipment in the event that the doors are inoperable.

- Broaden future seismic risk assessment efforts to include non-Port critical assets and lifelines, in coordination with other agencies and with utility owners.
- Confirm the plan for Port emergency operations and recovery. Immediate occupancy following any significant ground motion should not be expected for any Port facility as it currently exists. The Port should assess the current emergency response plan to ensure there is an allowance for the probable temporary unavailability of existing Port facilities.

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APPENDICES

Appendix 1: Study Team

Appendix 2: Geotechnical Report
Geotechnical Resources, Inc; New Albion Geotechnical

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ImageCat. Inc.

Appendix 1 – Study Team

Study Team

Port of Portland

The Port of Portland is charged with promoting aviation, maritime, commercial and industrial interests within Clackamas, Multnomah, and Washington counties (including the City of Portland). The scope of Port services extends beyond this immediate metropolitan area to include farmers and other industries from inland regions of the Northwest. The Port owns four marine terminals, Oregon's primary commercial airport (Portland International Airport), and two general aviation airports (Hillsboro and Troutdale), and oversees six industrial/business parks. The Port also owns and operates the dredge *OREGON* to help maintain the shipping channel on the lower Columbia River. More information at www.portofportland.com.

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HNTB Corporation – *Project Management, Report Compilation and Editing*

HNTB Corporation is an employee-owned, national infrastructure solutions firm serving public and private owners and contractors. With more than a century of service, HNTB understands the life cycle of infrastructure and has assisted clients with complex transportation and architecture infrastructure projects. HNTB's professionals nationwide deliver a full range of infrastructure-related services, including award-winning planning, design, program management, and construction management. HNTB's Pacific Northwest practice, which began 50 years ago, consists of more than 150 professionals in disciplines ranging from civil, structural, aviation, electrical and mechanical engineering to planning, architecture, and construction management. For more information, go to www.HNTB.com.

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BergerABAM – *Marine Facilities Engineering*

BergerABAM is a multidiscipline consulting firm that offers an exceptional portfolio of services in the areas of structural and civil engineering, environmental services, land-use planning, landscape architecture, and project and construction management. BergerABAM is best known in the industry for their port-related planning, permitting, and engineering work. BergerABAM has a 60-year history of providing these services to the port industry in the Pacific Northwest and worldwide. BergerABAM's waterfront group specializes in marine planning and engineering that includes waterfront and upland terminal facility planning and design, intermodal and multi-modal facility design, wharf and in-water structure condition assessments, and project cost estimating and scheduling. BergerABAM's unmatched expertise with waterfront facilities allows them to provide clients with maximum value and flexibility regarding their assets. More information at www.abam.com.

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Geotechnical Resources, Inc. – *Geotechnical Engineering*

GRI operates from offices in Beaverton and Brookings, Oregon, and Vancouver, Washington. The firm has a total staff of 33, with a technical staff of 28 geotechnical engineers and engineering geologists. The firm focuses on a wide range of challenging geotechnical and earthquake engineering projects with emphasis on the ports, energy, transportation, infrastructure, and building markets. The size and technical excellence of the staff permit GRI to offer a broad range of expertise, plus personalized service to each client. GRI has the practical, hands-on experience required to economically and accurately characterize site conditions and provide innovative, practical, and buildable recommendations for design and construction. More information at www.gri.com.

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ImageCat, Inc. – *Seismic Risk and Benefit-Cost Analyses*

ImageCat, Inc. is an international risk management company, developing new ways to meet global risk management needs using advanced computer-based and web-based technologies, together with engineering review and observation. ImageCat is headquartered in Long Beach, California, with a European office in London, England. Since its establishment in 2000, ImageCat has helped Government agencies, industry clients and research organizations prepare for and respond to disasters including earthquakes, hurricanes, and technological perils. Government clients include FEMA, U.S. Department of Transportation, the Federal Highway Administration, DARPA, NIST, and the California Governor's Office of Emergency Services. Research clients include the National Science Foundation, the Multidisciplinary Center for Earthquake Engineering Research, the Pacific Earthquake Engineering Research Center, and NASA. Insurance industry clients include Baseline Management and Applied Insurance Research (AIR).

More information about ImageCat is available at www.imagecatinc.com. More information about ImageCat's SeismiCat Seismic Risk Management System is available at www.seismicat.com.

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KPFF Consulting Engineers – *Building Structural Engineering*

KPFF provides innovative structural and civil engineering solutions for projects of all scales. Founded in 1960, KPFF is a leader in engineering and sustainable design, providing visionary, environmentally sensitive and technically advanced services to help maintain Oregon's special quality of life. KPFF approaches each design challenge as an opportunity to fulfill their passion for solving problems, delivering excellence and enabling the growth and creativity of their people, partners and profession. The firm has 15 office locations and employs over 800 people firm wide. KPFF's Portland office has a staff of 154. More information at www.kpff.com.

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New Albion Geotechnical, Inc. – *Geotechnical Engineering*

New Albion Geotechnical, Inc. provides technical specialization in geotechnical and earthquake engineering applications in support of maritime and surface transportation systems, major gas/liquid fuels and electrical power distributions systems, and coastal infrastructure. Specific areas of practice include: static and seismic stability of earth structures and earth retention systems, seismic and geologic hazard analyses and mitigation strategies using ground treatment, and performance-based seismic design and dynamic soil-structure interaction for pile supported structures, including major bridges, piers, and wharves. Dr. Stephen Dickenson, PE, D PE (Principal Engineer) has been active in geotechnical engineering research, education, and professional practice since 1985. His international consulting and applied research has focused primarily on geotechnical earthquake engineering applications addressing: seismic hazard evaluation, dynamic soil response, liquefaction evaluation and mitigation, slope stability and seismically-induced ground deformation, dynamic soil-foundation–structure interaction, and performance-based seismic design of maritime and surface transportation infrastructure.

Dr. Dickenson's active participation on numerous professional committees and panels has led to the development of guidelines and standards for the seismic design of port and lifeline infrastructure incorporating performance-based design principles for dynamic soil-foundation-structure interaction of port waterfront structures, specifically sheetpile bulkheads and pile-supported wharves. New Albion routinely provides professional engineers with continuing education courses and hands-on training on these various seismic design applications.

Additional background on the principal sectors served by New Albion and supporting professional resources can be obtained at www.newalbiongeotechnical.com.

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Appendix 2 – Geotechnical Report

(Geotechnical Resources, Inc and New Albion Geotechnical)

Geotechnical Consultation

Port of Portland Seismic Risk Assessment
HNTB Project No. 588-DS-002
Portland, Oregon

Prepared by

GRI
Oregon & Washington

NEW ALBION
Geotechnical, Inc.

February 23, 2015

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1.0 PROJECT DESCRIPTION

1.1 General

The Port is conducting a comprehensive seismic risk assessment of its key assets, which currently consist of approximately 20 structures located at the Portland International Airport (PDX), shipping terminals T-4 and T-5 along the Willamette River and T-6 along the Columbia River, and runways located at PDX and the Hillsboro Airport (HIO). The key assets addressed by the project team are discussed in detail in the comprehensive, final project report. The primary purpose of the project is to provide a screening-level evaluation of the seismic performance of these key Port structures and provide cost/benefit-based preliminary recommendations for possible mitigation of seismic risks. For this multi-asset Benefit-Cost Analysis (BCA), it should be emphasized that the study team focused on a level of geotechnical analyses sufficient to provide a preliminary assessment of seismic performance and identify structures and facilities that likely warrant additional, more refined, investigation in subsequent phases of the Port seismic risk reduction program.

The seismic risk assessment and financial modeling requires an integrated project approach that addresses the interrelated evaluation of seismic hazards, geotechnical earthquake engineering, and structural analysis and seismic performance assessment, all of which is synthesized into the port-wide seismic risk and financial model. GRI and NA have provided internal consultation, and seismic and geotechnical input (ground motions, ground deformations, ground treatment strategies and costs) to project team members that are addressing structural-related seismic risk issues. Structural engineering services for the building and waterfront structures are being provided by KPFF Consulting Engineers (KPFF) and BergerABAM, respectively, of Portland, Oregon. The fragility and financial risk evaluations for the project are being completed by ImageCat, Inc. of Long Beach, California. The results of our ground motion analyses have been used directly by all members of the team to evaluate seismic loading on the structures at the ARP of interest for the specific asset.

1.2 Seismic and Geotechnical Input for Integrated Seismic Performance Evaluation

Characterization of seismic ground motions serves as the basis for evaluating the performance of soils, foundations, and structures. Seismic events can result in two types of loading on structures: 1) inertial loading as the result of ground shaking, and 2) kinematic loading as the result of permanent soil displacement against foundations or other portions of a structure. Therefore, the seismic performance of embedded structures, pavements, underground utilities, and ancillary infrastructure is dependent on inertial and kinematic loading. To evaluate inertial loading on a structure, the anticipated seismic ground motions can be characterized in the form of a ground surface acceleration response spectrum. The evaluation of kinematic loading, however, requires several additional steps of geotechnical analysis: 1) evaluation of the potential for and possible extents of soil strength loss, or liquefaction; and 2) estimation of soil displacement as a result of strength loss, or liquefaction. Kinematic impacts to the structure are then estimated by the structural engineer using the anticipated soil displacements or forces.

GRI and NA have provided the project team with acceleration response spectra and estimated permanent soil displacements to evaluate inertial and kinematic loading, respectively. The analyses performed by GRI and NA have been performed using standard-of-practice procedures, using available geologic, geotechnical, and geophysical data from project files at GRI, NA, and the Port of Portland, and publicly available technical literature. Geotechnical site investigation, such as in situ testing, drilling and sampling, or laboratory testing, was not performed as a part of this consultation. The extent of the geotechnical data available for the priority asset sites varies substantially, and the conclusions and recommendations in this report reflect the assumptions and approximations necessary to provide the project team with input for structural analyses. The estimations of seismic ground motions, permanent ground deformations, and ground treatment strategies for mitigating liquefaction hazards are considered to provide reasonable ranges of anticipated seismic performance for a system-wide seismic risk investigation, the goal of which is to highlight primary vulnerabilities and identify key assets that warrant additional, more refined, site-specific geotechnical and structural investigations. The results of the seismic and geotechnical evaluations are intended to represent reasonable ranges of anticipated seismic behavior and performance at prescribed seismic hazard levels, and should be interpreted as consistent with an "advanced screening" level of analysis leading to subsequent site-specific analyses.

The impact of inertial loading on the priority asset structures has been evaluated by the project team using estimated ground surface response spectra prepared by GRI and NA for the various ARPs of interest. It should be noted that these ground surface response spectra were developed for use in fragility analyses of existing structures only and are not intended for design purposes. The estimated ground surface spectra represent "best estimates" of the anticipated ground motions at the project sites using trends from computed 1D dynamic site response analyses from sites at and near the Port facilities. In this regard, variation should be anticipated between our recommended spectra for fragility analysis and any code-based spectra for use in structural design which are developed with a squared-type spectrum and additional code limitations. As previously mentioned, extensive geotechnical data were not available for all of the priority asset sites, and the anticipated ground motions were approximated, where necessary, using data from local sites and judgment-based estimation based on extensive work by GRI, NA, and others at the Port of Portland.

The impact of kinematic loading associated with permanent ground deformation (PGD) has also been evaluated for the priority assets by the project team using the trends of "index" PGD versus ARP developed by GRI and NA. The vertical and lateral PGDs are considered "index" values in that they have been estimated using standard-of-practice engineering procedures in conjunction with currently available geotechnical data for each site. PGD trends were developed for free-field conditions and modified using simple scaling relationships to account for asset-specific soil-structure interaction for some of the pile-supported structures. More advanced analyses completed to evaluate the behavior of shallow foundation systems are discussed in Section 2.5.2 of this report. The estimated PGDs are considered applicable and reasonable for the current general seismic risk assessment; however, they should not be used as the basis for subsequent site-specific design of mitigation schemes. On the basis of the port-wide seismic risk assessment, we anticipate additional geotechnical investigations may be completed for some assets and more-refined analysis of dynamic soil-foundation-structure interaction performed for specific locations during subsequent phases of the Port seismic resiliency planning.

Information regarding inertial and kinematic loading has been provided to the other members of the team by NA and GRI throughout the course of the project. BergerABAM has used the index PGDs to evaluate the effects of kinematic soil loading on maritime structures, and KPFF has used the index PGDs to evaluate structural distress due to differential settlement beneath building structures and pavements.

1.3 Liquefaction Background

Liquefaction is a mechanism through which loose, saturated, granular materials, such as sand, and to a somewhat lesser degree soft, fine-grained soils such as non-plastic to low-plasticity silts, temporarily lose strength during and immediately after cyclic loading. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. As porewater pressure increases, the soil begins to lose strength, and may even temporarily behave as a viscous liquid in the most extreme cases. As strength is lost, there is an increased risk of settlement, and on sloping sites, also an increased risk of lateral spreading and/or slope instability. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

Methods for evaluating the triggering and consequences of liquefaction for granular soils such as sand are well established in practice, as sands have had a long case history of being susceptible to liquefaction. Practice-oriented methods to evaluate the cyclic behavior of fine-grained soils such as silt and clay when subjected to earthquake loading is not as well addressed in the technical literature, however, and the potential loss of strength and associated volumetric strain of fine-grained soils has become a significant topic of research. GRI and NA have compiled an extensive collection of laboratory test data for characterizing the cyclic behavior of local silts and have significant experience evaluating silt liquefaction and post-cyclic behavior.

2.0 APPROACH AND ANALYSES

2.1 General

The requisite first step in the seismic risk assessment for all assets is the characterization of the ground motions to be used by the project team members. The ground motions have been estimated as functions of the ARP, or inversely the Annual Frequency of Occurrence, and provided to the project team in the form of acceleration response spectra. To develop trends as a function of ARP, site response, liquefaction, and liquefaction-induced ground deformation were evaluated for the following five hazard levels, i.e., ARP: 72 years, 224 years, 475 years, 975 years, and 2,475 years. This information was used by the project team to evaluate the influence of inertial and kinematic loading on structural performance. The following sections of this report provide a brief summary of our approach to the different analyses.

2.2 Seismic Hazard Evaluation and Characterization of Seismic Motions on Bedrock

To evaluate regional seismic hazard and characterize bedrock ground motions at the locations of the various priority assets, we reviewed the results of the 2008 and 2014 U.S. Geologic Survey (USGS) Probabilistic Seismic Hazard Analyses (PSHA). For a given site location and ARP, the PSHA provides estimates of the ground motions on bedrock (Site Class B/C boundary) in terms of response spectral ordinates based on a probabilistic evaluation of the spatial and temporal occurrence of earthquakes throughout the region. The bedrock motions estimated by the PSHA are presented in the form of the Uniform Hazard Spectrum (UHS), which serves as the basis for many design standards. The results of the PSHA also provide detailed information regarding the predominant seismic sources that contribute to the ground motions at the selected ARPs. This process, referred to as seismic hazard "deaggregation," is necessary for subsequent analyses, such as liquefaction susceptibility and earthquake-induced permanent ground deformation. GRI and NA have used the results of the 2008 USGS PSHA and deaggregation as the basis for the ground motion characterization. The 0-, 0.2-, and 1.0-second spectral ground motions on bedrock/firm base (Site Class B/C boundary) conditions are shown on Figure 2-1.

2.3 Dynamic Soil Response and Site Effects

The seismic performance of the priority asset structures is dependent on the anticipated seismic motions at or very near the ground surface as opposed to the seismic motions on bedrock. The anticipated ground surface motions are greatly influenced by soil conditions, and dynamic soil response analysis is commonly performed to estimate the site-specific influence of a soil column on bedrock ground motions. The site-specific influence of soils on bedrock motions is typically quantified using a Spectral Amplification Ratio (SAR), which is defined as the ratio of the ground surface to bedrock seismic motions at a given spectral period. The SAR is a function of soil stratigraphy (types and thickness), soil stiffness and cyclic behavior, as well as the amplitude of the bedrock motions. It should be noted that the SAR can be greater or less than unity, demonstrating an increase or decrease, respectively, in the amplitude of the bedrock motions at a given ARP.

To streamline the evaluation of dynamic soil response and make full use of numerous previous analyses, GRI and NA estimated the SARs at the ARPs of interest based on three routine methods: 1) using current Ground Motion Prediction Equations (GMPE) that account for, in a simple manner, the influence of soils in the upper 100 ft (30 m) of the profile on the characteristics of the motions; 2) consideration of the spectral amplification factors provided in current building codes and standards, which are also based on soil characteristics in the upper 100 ft of the profile; and 3) trends in amplification from site-specific modeling performed by GRI, NA, and others. The site-specific approach is considered to be the most relevant for this project given the deep soil profiles that are present at the asset locations and the inherent inability of the other methods to account for soils below a depth of 100 ft.

The approach to evaluating site response for this project consisted of the following specific steps:

1) Compile site-specific shear wave velocity (Vs) data for the Port assets. This data was used to compute the average shear wave velocity in the upper 100 ft of the soil column, or $(Vs)_{30}$, for all sites at which data were available. The results of this compilation of shear wave velocity data is shown on Figure 2-2. It is apparent from the available in situ geophysical data that with the exception of the Hillsboro Airport site, the $(V_s)_{30}$ values fall into a relatively narrow band, and an average value of 600 ft/sec can be used to characterize the soil profiles at the Port asset sites for the sake of estimating dynamic soil response using simplified, practice-oriented procedures.

2) Review and compile the results of several available site-specific, free-field, one-dimensional site-response analyses completed at and in the vicinity of the Port assets by GRI, NA, and others. It is noted that all of the dynamic response analyses reviewed for this project were performed using the equivalent linear, total stress model SHAKE.

3) Using the results of the site-response analyses, develop Port-specific trends for soil amplification/de-amplification of bedrock motions as functions of input shaking level (i.e., ARP or Seismic Hazard Level). As previously discussed, soil amplification/de-amplification is most commonly described using a spectral amplification ratio (SAR),

which is defined as the ratio of the spectral ordinates for ground surface to bedrock motions at the period of interest. The SAR was specifically evaluated at four discrete oscillator periods of interest: 0.0 second (PGA), 0.2 second, 0.5 second, and 1.0 second. From these four data points, the complete acceleration response spectra were developed. The compiled SAR values at each oscillator period were found to define relatively uniform trends with ground motion level (i.e., ARP) at the Port sites for which modeling had been performed. This supported the use of a uniform set of SAR curves at PDX and maritime asset sites and a separate set of site-specific SAR curves at the Hillsboro Airport.

4) Estimate the site-specific ground surface response spectra at each ARP of interest by multiplying the 2008 USGS Site Class B/C UHS values by the Port specific SAR trends.

5) Compare the site-specific response spectra with: 1) the 2008 USGS ground surface spectra based on GMPEs and Port-specific V_{S30} data, and 2) the ASCE 7-10 code-based ground surface spectra. Our recommended site-specific spectra for fragility analyses were based on a weighted averaging of the trends developed using the site-specific dynamic modeling and ASCE 7-10 Site Class D/E values, with weighting factors of two-thirds and one-third, respectively. The comparison plots, which include the recommended spectra for fragility analyses, are provided on Figures 2-3 through 2-7. The recommended 0-, 0.2-, and 1.0-second spectral ground surface motions are also summarized on Figure 2-8.

6) As shown on Figures 2-3 through 2-7, the recommended ground motions for fragility analysis are typically lower than code-based and USGS values. As previously discussed, the recommend motions for fragility analysis represent "best-estimate" trends for structural performance assessment and are not intended for design purposes, which require adherence to applicable standards and codes. In this regard, the resulting ground surface motions for fragility analysis do not necessarily match the ground motions developed in accordance with ASCE 7-10 or ASCE 41-13.

2.4 Liquefaction Hazard Evaluation (Triggering)

Our approach to the site-specific liquefaction evaluation of each Port asset using available subsurface information consisted of the following steps:

1) Review available site-specific and local subsurface information. Subsurface information was provided by the Port and supplemented with information from our project files.

2) Create representative soil profiles at each asset using the results of the most relevant explorations. For sites with variable subsurface conditions, several different profiles were considered in the evaluations.

3) Based on a review of the 2008 USGS PSHA deaggregations, estimate the ground surface peak horizontal acceleration (PGA) for the predominant seismic sources,

evaluated based on earthquake magnitude, source-to-site distance, and Port-specific SAR trends, at the five discrete hazard levels.

4) The standard of practice methodology for liquefaction analyses requires consideration of both PGA and earthquake magnitude. The USGS PSHA-derived PGA values are associated with the uniform hazard and therefore consider the cumulative impact of all seismic sources that are considered a hazard to the site. A thorough site-specific evaluation of liquefaction hazard requires identification of the primary seismic sources at a given ARP, determination of their relative contribution to the uniform hazard, and estimation of the PGA associated with the specific seismic source in accordance with the USGS PSHA from which the deaggregation was performed. In the Portland region, the PSHA demonstrates that at each ARP of interest, as many as five or six seismic sources significantly contribute the overall seismic hazard. It is common practice to select two or three of the primary seismic sources and perform liquefaction triggering analyses for each source using a representative PGA value for the given earthquake magnitude and source-to-site distance. This procedure was precluded on the port-wide seismic risk assessment due to the large number of assets, the number of ARPs required by the project team at each asset (five), and the number of primary seismic sources identified for each ARP. For this reason, a commonly used method of PGA scaling for each of the primary seismic sources was implemented to streamline the site-specific liquefaction hazard analyses performed for each asset. This method is described as follows.

   a) For each asset and ARP, a single, representative value of PGA for a reference earthquake magnitude was estimated based on the PSHA deaggregation and the relative contribution of the individual seismic sources. In general, for each ARP, the four to six primary seismic sources were identified and tabulated with their respective magnitude and estimated mean PGA. The estimated PGA values were then normalized to account for the influence of earthquake duration using the Magnitude Scaling Factor (MSF) of Idriss and Boulanger (2008). For each primary source, the magnitude-normalized PGA value (PGA/MSF) was then multiplied by the corresponding percentage contribution to the overall seismic hazard to cumulatively estimate a single, representatively weighted value of normalized PGA. The single, representative value of normalized PGA was then anchored to a magnitude of 7.5, consistent with the development of the methods for evaluating liquefaction triggering. At this magnitude the MSF is equal to unity (1.0).

      This procedure has been widely used in regional liquefaction hazard mapping projects and it clearly illustrates the relative importance of the individual seismic sources on the overall liquefaction susceptibility at a site for a given ARP. This method is advantageous for this project for the following reasons;

      i) It is based on standard of practice procedures for evaluating liquefaction hazard using site-specific geotechnical data.

ii) The relative influence of the various seismic sources (magnitude and source-to-site distance pairs) can be easily determined.

iii) The applicability of a single value of PGA for a reference magnitude of 7.5 can be assessed in terms of a level of conservatism in the liquefaction hazard assessment.

iv) The use of a single value of PGA for the liquefaction hazard assessment for each asset and ARP greatly increases computational efficiency for a system-wide hazard evaluation.

5) The HIO Runway 13-31 was evaluated only for a deterministic or single specified M9.0 Cascadia Subduction Zone (CSZ) earthquake scenario. In accordance with the 2008 USGS PSHA, the PGA for the M9.0 CSZ event was estimated using the GMPEs of Atkinson and Boore (2003), Youngs, et al. (1997), and Zhao, et al. (2006) with a 25%, 25%, and 50% weighting, respectively. .

6) Perform site-specific liquefaction evaluations using the procedures presented by Idriss and Boulanger (2008). For sandy soils, the cyclic resistance to liquefaction is typically evaluated based on in situ testing by either the Standard Penetration Test (SPT N-values) or Cone Penetration Test (CPT Qc values) with corrections for fines content. For silty and clayey soils, additional factors such as consolidation stress history and/or the results of more advanced laboratory tests have been used to evaluate soil resistance to liquefaction. A large collection of laboratory test data compiled by NA and GRI for local silt soils was used to supplement the cyclic resistance assessment of the local fine-grained soils.

2.5 Liquefaction-Induced Ground Deformation

2.5.1 General

Liquefaction-induced, free-field permanent ground deformations (PGD) were evaluated for both vertical (PGD_V) and horizontal movements (PGD_H). Our approach to estimating liquefaction-induced ground deformations consisted of the following steps.

2.5.2 One-Dimensional Ground Surface Settlement (Index PGD_V)

1) The results of the liquefaction triggering evaluations were used to identify zones within the soil profile at each asset location that are susceptible to the triggering and surface manifestation of liquefaction for a given ARP.

2) Estimate free-field liquefaction-induced vertical deformations due to post-cyclic loading reconsolidation at the building assets and upland portions of the waterfront assets using the 1-D laboratory-based method presented by Ishihara and Yoshimine (1992), with reduction factors based on field case history investigations as recommended by Tsukamoto and Ishihara (2010). A cutoff depth of 60 ft was implemented in the analyses based on recommendations provided by Cetin, et al.

(2009), which indicate the effects of liquefaction-induced settlement occurring below this depth are typically not observed at the ground surface.

3) For sandy soils, the 1-D volumetric settlements were estimated using relationships based on SPT N-values as developed by Ishihara and Yoshimine (1992) and recommended by Idriss and Boulanger (2008). For silty and clayey soils, the 1-D volumetric settlements were estimated using laboratory-based procedures specifically developed for fine-grained soils in the Portland region. In general, the vertical strain potential for silty and clayey soils is significantly lower than sandy soils, and, for a given factor of safety against cyclic degradation or liquefaction, these soils typically exhibit less post-earthquake settlement than sands.

4) For structures supported by shallow foundations, evaluate the potential for additional liquefaction-induced footing settlement due to reduced bearing capacity and mobilization of shear strain beneath the footing (deviatoric settlement) using the methods of Naesgaard, et al. (1998) with additional recommendations provided by the Greater Vancouver Liquefaction Task Force (Anderson et al., 2007). In these methods, the settlement of shallow foundations is estimated based on the thickness and strength characteristics of both the non-liquefied soil "crust" and the underlying liquefied soils beneath the footing. The amount of settlement estimated using these methods is due to the applied bearing stress beneath the footing and is considered additive to the estimated volumetric settlement. It should also be noted that liquefaction-induced footing settlement is likely to occur during seismic loading, whereas liquefaction-induced free-field settlement occurs immediately after the seismic event.

5) For structures supported by deep foundations, evaluate the potential for additional liquefaction-induced pile settlement or loss of bearing capacity. The settlement of deep foundations due to significant excess pore pressure generation along portions of the pile was estimated based on historical observations and tempered by engineering judgment. In most cases, the deep foundation settlements were estimated as a proportion of the free-field ground settlement, taking into consideration the extent of the liquefied zone along the pile and the depth of embedment of the pile tip into non-liquefiable soils.

6) Using the results of the deformation analyses, develop a trend for Index PGD_V versus ARP for the aviation assets and upland portions of the waterfront assets. Where appropriate, both free-field displacements and the estimated displacement of the structure are provided.

2.5.3 Horizontal Ground Deformation (Index PGD_H)

1) The results of the liquefaction triggering evaluations were used to identify zones within the soil profile at each asset location that are susceptible to liquefaction triggering and demonstrate a potential for shear strain mobilization resulting in lateral spreading displacements during and immediately following seismic loading. As an example, Figure 2-9 shows a typical maritime cross section illustrating zones of potentially

liquefiable soil at a specific hazard level of 475 years. Using this general approach, the soil profiles of interest were evaluated at the five discrete ARPs to estimate the extents of liquefaction and liquefaction-induced lateral spreading at the asset sites for each ARP.

2) Estimate liquefaction-induced horizontal deformations in sandy soils using two, practice-oriented procedures as follows, 1) the simple, empirically based method presented by Youd, et al. (2002), and 2) a CPT-based procedure developed by Zhang, et al. (2004) that correlates the cyclic shear strain potential of sandy soils to in situ measurements based on the results of laboratory testing. Both methods were used at each asset located in the vicinity of a slope or stream channel face, with an equal weighting of the two methods used to define the index PGD_H at each ARP. The weighting procedure was used in an attempt to reduce the influence of acknowledged limitations and assumptions in each method on the final trend of index PGD_H with ARP. For evaluations using the method of Zhang, et al. (2004), cumulative lateral deformation was estimated from the top of the liquefiable layers down to the elevation of the base of the liquefiable layer(s) or the dredge depth of the river, whichever was less. Deep-seated failures extending below the base of the channel were not explicitly considered in the deformation estimates. However, deep-seated failures are inherently considered in the empirical method of Youd, et al. (2002), and it has been acknowledged (Kramer, 2008) that the procedure of Zhang, et al. (2004) provides an estimate that is more closely related to the maximum range of potential lateral deformation as opposed to the "best-estimate" mean displacement. Therefore, these methods may provide adequate estimates for cases where failures extend to a small to moderate depth beneath the dredge line. For silty and clayey soils, compare the horizontal deformations estimated by the Zhang, et al. (2004) methodology with soil-specific ranges of "limiting" maximum shear strain potential established through an extensive review of cyclic Direct Simple Shear test data on local soils. The lateral strain potential for silty and clayey soils is significantly lower than sands, and for a given factor of safety against liquefaction or cyclic degradation, silty and clayey soils exhibit less seismically induced horizontal deformation than sands. In cases where the estimated values of seismic ally induced shear strain exceed the soil-specific limits, the limiting values were implemented in the calculations.

3) Using the results of the deformation analyses, develop a trend for Index PGD_H versus ARP for the waterfront assets. Where appropriate, both free-field displacements and the estimated displacement of the structure are provided.

3.0 RESULTS AND ASSET SUMMARY

3.1 General

For the purpose of discussion, the results of our evaluations and an overall summary for each priority asset are presented in tabular form in the following sections. Each table provides a brief description of the asset being evaluated, an overview of the subsurface conditions at the asset site, a list of geotechnical reports used to generate representative soil profiles for analysis, and a discussion of and reference to the PGD plots generated for the project. At the end of each section, a preliminary discussion of possible mitigation alternatives is provided.

3.2 Maritime Assets

3.2.1 General

The maritime assets evaluated for this project consist of Port identified priority dock structures located at shipping terminals T-4 and T-5 along the Willamette River and T-6 along the Columbia River. In most cases, permanent vertical ground deformations were evaluated using geotechnical data and site configuration representative of the upland portions of the site, and permanent horizontal deformations were evaluated at riverbank or overwater portions of the site. For these evaluations, the groundwater level at the sites was assumed to coincide with river level. It should be noted that many geotechnical reports and supplementary technical documents that are not referenced herein were reviewed to assist in evaluating the variability in subsurface conditions at each asset location.

Berth 410	
Structure Type	Pile-supported dock with timber piles. See BergerABAM report for additional information.
Subsurface Conditions	The soil profile at the mudline consists of approximately 20 ft of soft to medium stiff, normally consolidated, low plasticity silt underlain by loose to medium dense sand to the maximum depth considered in the analyses.
Referenced Report	Blasland, Bouck, & Lee, 2004 (overwater explorations)
PGD_V	N/A.
PGD_H	Figure 3-1. Applies to areas downslope of the shoreline and accounts for the presence of dock piles.

Berth 411	
Structure Type	Pile-supported dock with reinforced concrete piles, quay wall, and sheet pile toe wall. See BergerABAM report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of interbedded layers of loose to medium dense sand and soft to medium stiff, normally consolidated to slightly overconsolidated, low-plasticity silt.
Referenced Reports	Blasland, Bouck, & Lee, 2004 (overwater explorations) L.R. Squier Associates, Inc., 1984 (upland explorations) Engineering Systems Solutions, 2013 (upland explorations)
PGD_V	Figure 3-2. Applies to upland areas adjacent to the dock.
PGD_H	Figure 3-2. Applies to dock areas and accounts for the presence of dock piles.

Berth 501	
Structure Type	Cellular bulkhead dock and pile-supported access trestle. See BergerABAM report for additional information.
Subsurface Conditions	The soil profile near the riverbank at the site typically consists of interbedded layers of loose to medium dense sand and very soft to medium stiff, lightly to moderately overconsolidated, moderate- to high-plasticity silt. The soil profile in the upland portions of the site typically contains more silt.
Referenced Reports	GRI, 1988 (Berth 502 upland explorations) GRI, 1989 (Berth 502 upland explorations) GeoDesign, Inc., 1998 (upland explorations)
PGD_V	Figure 3-3. Applies to upland areas of the site near the trestle tower and grain silos.
PGD_H	Figure 3-3. Applies to areas downslope of the shoreline.

Berth 503	
Structure Type	Pile-supported dock and trestle with steel pipe piles. See BergerABAM report for additional information.
Subsurface Conditions	The soil profile in the overwater portions of the site typically consist of about 10 ft of very soft to soft, slightly overconsolidated, low-plasticity silt underlain by loose to medium dense sand to the maximum depth considered in the analyses. The soil profile in the upland portions of the site typically consists of about 40 ft of soft to medium stiff, lightly to moderately overconsolidated, low-plasticity silt underlain by medium dense sand to the maximum depth considered.
Referenced Report	Geocon Northwest, Inc., 2012 (upland explorations) Foundation Sciences, Inc., 1981 (overwater explorations)
PGD_V	Figure 3-4. Applies to upland areas of the site near the administration buildings and elevated product trestle foundations.
PGD_H	Figure 3-4. Applies to upland areas adjacent to the riverbank and to locations downslope of the shoreline.

Berth 601	
Structure Type	Floating dock and pile-supported trestle with steel pipe piles. See BergerABAM report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of loose to dense sand with occasional layers of very soft to medium stiff, lightly to moderately overconsolidated, low plasticity silt.
Referenced Reports	GRI, 1989 (upland and overwater explorations) GeoEngineers, 2010 (upland explorations)
PGD_V	Figure 3-5. Applies to upland areas of the site near the access trestle.
PGD_H	Figure 3-5. Applies to areas from the shoreline to the floating dock, namely trestle foundations and dolphin piles.

3.2.2 Possible Mitigation Alternatives for Maritime Structures

Based on discussions with BergerABAM, it is anticipated the maritime asset structures cannot tolerate the estimated permanent ground deformations at most hazard levels considered and remain operational. In support of the overall seismic risk evaluation and cost-benefit assessment for mitigation strategies at the selected Port assets we have developed preliminary recommendations for ground improvement at the maritime sites to limit permanent soil deformations and associated soil loading on the dock structures. Due to common permitting constraints, at this time, we have assumed ground improvement can only be implemented above the ordinary high water level. In our opinion, stone columns will provide the most economical and constructible alternative for ground improvement at the sites. Based on our discussions with local ground improvement contractors, we anticipate the stone columns will cost approximately $40 per vertical lineal ft based on a treatment depth of about 80 ft. Assuming a center-to-center column spacing of 8 ft and a treatment width of approximately 60 ft, we anticipate the ground improvement will cost on the order of $3,000 per lineal ft of riverbank.

3.3 Aviation Assets – Structures

3.3.1 General

The aviation asset structures evaluated for this project typically consist of a wide range of single- and multi-story buildings located at PDX and T-6 on the Columbia River. For these structures, only permanent vertical ground deformations were evaluated due to the absence of significantly sloping ground at the asset locations. It should be noted that many geotechnical reports that are not referenced herein were reviewed to assist in evaluating the variability in subsurface conditions at each asset location.

T-6 CDC Warehouse and Electrical Shop	
Structure Type:	Single-story buildings with shallow foundation systems. See KPFF report for additional information.
Subsurface Conditions:	The soil profile at the site typically consists of loose to medium dense sand with occasional layers of medium stiff, slightly to moderately overconsolidated, low plasticity silt. Groundwater was assumed at depths of about 12 ft below the ground surface.
Referenced Report:	GRI, 1986
PGD_V:	Figure 3-6. Figure also provides discussion of estimated foundation settlements.
PGD_H	N/A

PDX ARFF	
Structure Type	Single-story building with mat foundation systems. See KPFF report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of loose to medium dense sand with occasional layers of soft to medium stiff, slightly to moderately overconsolidated, low plasticity silt on the order of 10 ft thick. Groundwater was assumed at a depth of about 8 ft below the ground surface.
Referenced Report	GeoEngineers, 2014
PGD_V	Figure 3-7. Figure also represents estimated foundation settlement.
PGD_H	N/A

PDX Concourse C	
Structure Type	Multi-story buildings with shallow and deep foundation systems. See KPFF report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of about 5 to 15 ft of loose to medium dense sand underlain by soft to medium stiff, slightly to moderately overconsolidated, low-plasticity silt to depths of 40 to 60 ft. Medium dense to dense sand is present beneath the silt. Groundwater was assumed at a depth of about 7 ft below the ground surface.
Referenced Reports	GRI, 1996 GRI, 1998
PGD_V	Figure 3-8. Figure also provides estimated pile-supported foundation settlement.
PGD_H	N/A

PDX Main Passenger Terminal – Southern Portion	
Structure Type	Multi-story buildings supported by wide range of shallow and deep foundation systems. See KPFF report for additional information.
Subsurface Conditions	The soil profile in the southern portions of the site typically consists of interbedded layers of loose to medium dense sand and very soft to medium stiff, slightly to moderately overconsolidated, low-plasticity silt to a depth of about 60 ft. The sand and silt layers are on the order of 10 to 15 ft thick. Medium dense to dense sand is present below a depth of about 60 ft. Groundwater was assumed at a depth of about 7 ft below the ground surface.
Referenced Report	GRI, 1998
PGD_V	Figure 3-9. Figure also provides estimated pile-supported foundation settlement for various pile lengths.
PGD_H	N/A

PDX HQ/P2	
Structure Type	Multi-story building supported by steel pipe piles. See KPFF report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of interbedded layers of loose to medium dense sand and soft to medium stiff, slightly to moderately overconsolidated, low-plasticity silt to depths up to about 90 ft. The sand and silt layers are on the order of 10 to 20 ft thick. Medium dense to dense sand is present below a depth of about 90 ft. Groundwater was assumed at a depth of about 7 ft below the ground surface.
Referenced Reports	GRI, 1987 GRI, 2006
PGD_V	Figure 3-10. Figure also provides estimated pile-supported foundation settlement.
PGD_H	N/A

PDX CUP	
Structure Type	Multi-story building supported by shallow and deep foundation systems. See KPFF report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of about 25 ft of very loose to medium dense sand over predominantly very soft to medium stiff, slightly to moderately overconsolidated, low-plasticity silt to a depth of about 60 ft, where medium dense to dense sands are present to the maximum depth considered in the analyses. Groundwater was assumed at a depth of about 7 ft below the ground surface.
Referenced Report	GRI, 2006
PGD_V	Figure 3-11. Figure also provides estimated pile-supported foundation settlement.
PGD_H	N/A

PDX Ground Maintenance and Administration Buildings	
Structure Type	Single-story buildings supported by shallow foundations systems. See KPFF report for additional information.
Subsurface Conditions	The soil profile at the site typically consists of loose to medium dense sand to a depth of about 50 ft and dense sand below this depth. Groundwater was assumed at a depth of about 8 ft below the ground surface.
Referenced Report	Northwest Testing Laboratories, 1983
PGD_V	Figure 3-12. Figure also provides estimated shallow foundation settlements.
PGD_H	N/A

3.3.2 Possible Mitigation Alternatives for Aviation Structures

Based on our discussions with KPFF, we anticipate many of the older aviation structures with shorter piles, spread foundations, or non-structurally supported floor slabs will not be fully operational following a significant seismic event without mitigating liquefaction hazards and/or structurally retrofit. Ground improvement mitigation using compaction or jet grouting techniques as well as structural retrofit using micropiles were both considered during preliminary stages of this project. For this level of analysis, KPFF has based their mitigation strategies on micropile foundation retrofits, which have been successfully used for several projects at PDX. Considering access and operational constraints in many of the buildings, micropile retrofits will likely require installation using low-overhead equipment.

3.4 Aviation Assets – Pavements

3.4.1 General

In accordance with the scope of work, the project team evaluated the anticipated seismic performance of the south runway at PDX and the main runway at HIO. The seismic performance of the main runway at HIO (runway 13R-31L) was evaluated for a single deterministic M9.0 CSZ earthquake scenario, and the south runway at PDX was evaluated at the five probabilistic hazard levels used for the remainder of this study. As discussed in Section 3.4.2, the north runway was also considered as part of a mitigation strategy.

For the south runway at PDX and the main runway at HIO, only permanent vertical ground deformations were evaluated due to the absence of significantly sloping ground at the sites. Based on information provided by the design team, the Federal Aviation Administration (FAA) considers up to 3 in. of vertical offset along runway pavements acceptable. Differential settlement caused by soil liquefaction is typically assumed to be on the order of half the total magnitude of liquefaction-induced settlement for most sites. Following this assumption, up to 6 in. of total liquefaction-induced settlement at the aviation sites may result in acceptable runway performance.

The results of the screening-level analyses indicate there is a low risk that HIO runway 13R-31L will exceed the allowable deformation criteria outlined above in response to the M9.0 CSZ earthquake scenario. While some settlement is expected, we anticipate the runway will be functional for emergency response purposes. It should be assumed that portions of the runway will need to be rebuilt for long-term commercial aviation use.

Analyses completed for the PDX south runway indicate there is a much higher risk of liquefaction-induced settlement and associated differential settlement, resulting in significantly more pavement damage than at HIO. Based on the results of the preliminary analyses for the PDX south runway and the FAA recommendations, we anticipate unacceptable differential settlements will occur at ARPs greater than approximately 225 years if the liquefaction hazard is not mitigated. A portion of this unacceptable settlement is attributed to the increased risks associated with differential settlement of the individual concrete pavements panels. We have also assumed the 3 in. of vertical offset specified by the FAA may be non-conservative, except for emergency, military-type aircraft.

PDX South Runway	
Structure Type:	Pavement.
Subsurface Conditions:	The soil profile along the alignment of the south runway typically consist of about 10 ft of loose to medium dense sand underlain by very soft to medium stiff, slightly to moderately overconsolidated, low-plasticity silt to a depth of about 50 or 60 ft. Medium dense to dense sand is present beneath the silt. Groundwater was assumed at a depth of about 8 ft below the ground surface. It should be noted that the available information for the south runway is limited to relatively shallow explorations, and the deeper portions of the soil profile were extrapolated from nearby explorations. For the purpose of these evaluations, we have assumed similar subsurface conditions and seismic performance for the north runway, which was not explicitly evaluated for this project.
Referenced Reports:	GRI, 1996 (concourse C) GRI, 1998 (concourse C) Cornforth Consultants, 2009 (south runway alignment)
PGD_V:	Figure 3-13. We anticipate up to half the total settlement may occur as differential settlement along the length of the runways, particularly in areas where containment dikes were constructed in conjunction with historic fill placement at the site. Due to the lack of deeper subsurface information along the runway alignments, further evaluation and quantification of differential settlements should be completed with additional geotechnical explorations.
PGD_H	N/A

HIO Runway 13-31	
Structure Type:	Pavement.
Subsurface Conditions:	The soil profile along the alignment of the runway 13-31 is variable in the upper 35 ft of depth. In the northern portions of the site, the upper 35 ft of the soil profile typically consists of soft to medium stiff, moderately to highly overconsolidated, low plasticity silt. In the central and southern portions of the site; however, interbedded layers of silty and clayey sand with varying thicknesses are present from depths of about 10 to 35 ft. The assumed subsurface conditions are relatively uniform below a depth of about 35 ft, consisting mainly of stiff, low to moderate plasticity silt that is not considered susceptible to seismically induced strength loss. Groundwater was assumed at a depth of about 10 ft below the ground surface. It should be noted the majority of the available subsurface information at the site was obtained from shallow borings completed off the runway alignment, and the representative profiles were supplemented with deeper subsurface information in the site vicinity. Due to the large variability in subsurface conditions along the length of the runway, three individual soil profiles were analyzed as part of our evaluation.
Referenced Reports:	F.M. Fox & Associates, Inc., 1974 (near south end of runway) Dames and Moore, 1986 (near south end of runway) GRI, 1993 (near middle of runway) GeoDesign, Inc., 1999 (near middle of runway) Kleinfelder, 2008 (near taxiway C) Kleinfelder, 2010 (near north end of runway) Kleinfelder, 2011 (near taxiway C)
PGD_V:	The results of our evaluations indicate the layers of silty sand are susceptible to liquefaction during the deterministic M9.0 earthquake scenario. We anticipate liquefaction-induced settlement near the north end of the runway will be within acceptable differential settlement limits. Near the intersection with Taxiway C and at the south end of the runway; however, we anticipate up to about 3.5 and 5.0 in. of liquefaction-induced settlement, respectively, may occur following the deterministic M9.0 event.
PGD_H	N/A

3.4.2 Possible Mitigation Alternatives for PDX Aviation Pavements

Based on the preliminary results presented during the course of this study, the Port requested the team evaluate three primary mitigation strategies for the south and north runways at PDX. Although the north runway was not specifically analyzed, the team made simplifying assumptions that the behavior would be somewhat similar to the south runway with the exception of an increased risk of lateral spreading at higher hazard levels. The three requested alternatives are summarized below:

1) **Jet Grouting of the PCC-paved South Runway**

   This scenario assumes several extended closures to complete mitigation without a complete repaving of the runway.

2) **Stone Column Mitigation of the PCC-paved South Runway**

   This scenario assumes mitigation will be completed during a future repaving of the runway.

3) **Stone Column Mitigation of the AC-paved North Runway**

   This scenario assumes mitigation will be completed during a future repaving of the runway.

Options 2 and 3 were evaluated because of the recent paving with PCC and the much longer anticipated design life relative to the current AC pavement at the north runway.

It should be noted that the preliminary ground improvement schemes are targeted at mitigation at the 975-year hazard level with acceptance of additional risk at greater hazard levels. The results of the preliminary analyses indicate jet grout and stone column ground improvement to a depth of about 30 and 40 ft, respectively, would likely limit liquefaction-induced ground surface settlements to tolerable levels.

Based on our discussions with local ground improvement contractors, we anticipate stone columns will cost approximately $35 per vertical linear ft based on a treatment depth of about 40 ft. Assuming a center-to-center column spacing of 8 ft and a 20-ft treatment margin around the south runway footprint, we anticipate the cost of stone column ground improvement for the south runway will be on the order of $67 million. Assuming the same parameters for the north runway, but implementing a 50-ft treatment margin along the northern perimeter to limit the risk of lateral deformation, we anticipate the cost of stone column ground improvement for the north runway will be on the order of $68 million. For the jet grout alternative for the south runway, we anticipate the columns will be installed to a depth of about 30 ft using an area replacement ratio of 20%, resulting in a total cost on the order of $137 million.

4.0 LIMITATIONS

The geotechnical and seismic hazard evaluations provided in support of the port-wide seismic risk assessment have required the collection, synthesis, and completion of subsequent analyses using data from numerous technical documents prepared by several organizations over a roughly 50-year timespan. Reports from the project files and archives at GRI and NA were supplemented to a large degree with reports by other consultants made available to the project team by the Port of Portland. GRI and NA did not participate in the implementation of the work by other consultants and did not independently verify the accuracy or completeness of the information contained in the reports. No warranty, either expressed or implied, is provided.

The scope of this assessment was limited to a review of existing geotechnical information, much of which was completed by others. No additional geotechnical investigations were performed as part of this project. Therefore, all of the analyses are the products of desktop studies using existing data and reflect uncertainties inherent in this type of study. For several of the selected assets, the base of geologic and geotechnical data is limited, necessitating the use of local trends in stratigraphy and geotechnical soil properties.

The scope of the seismic and geotechnical analyses was tailored with input from the project team, in consultation with the Port of Portland. The level of analytical rigor was commensurate with the primary goals of assessing the approximate cost-benefit relationships for implementation of mitigation strategies at the selected assets. As such, approximations and estimates are inherent in an "advanced screening" study of this type. The resulting seismic ally induced ground deformations are presented as "index" values that reflect necessary approximations and assumptions. While regionally accepted methods and standards of practice were used, in many cases the level of geotechnical site investigation was not adequate at a specific location to prepare any more than an approximate estimate of the index ground deformation. The primary goal of this preliminary level of seismic analysis is to contribute to the subsequent structural performance assessments and overall seismic risk assessment leading to the identification of key assets that would benefit from further, and more refined, site-specific evaluations. For these critical assets, it is anticipated that additional site investigations and engineering analyses would be conducted in support of potential mitigation and retrofit strategies, and more detailed cost-benefit analyses of individual facilities. These supplementary efforts are recommended as possible goals of a subsequent phase of work on specific assets, the scope of which would be guided using the results of this investigation.

Submitted for GRI,

Renews 6/2016

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This document has been submitted electronically.

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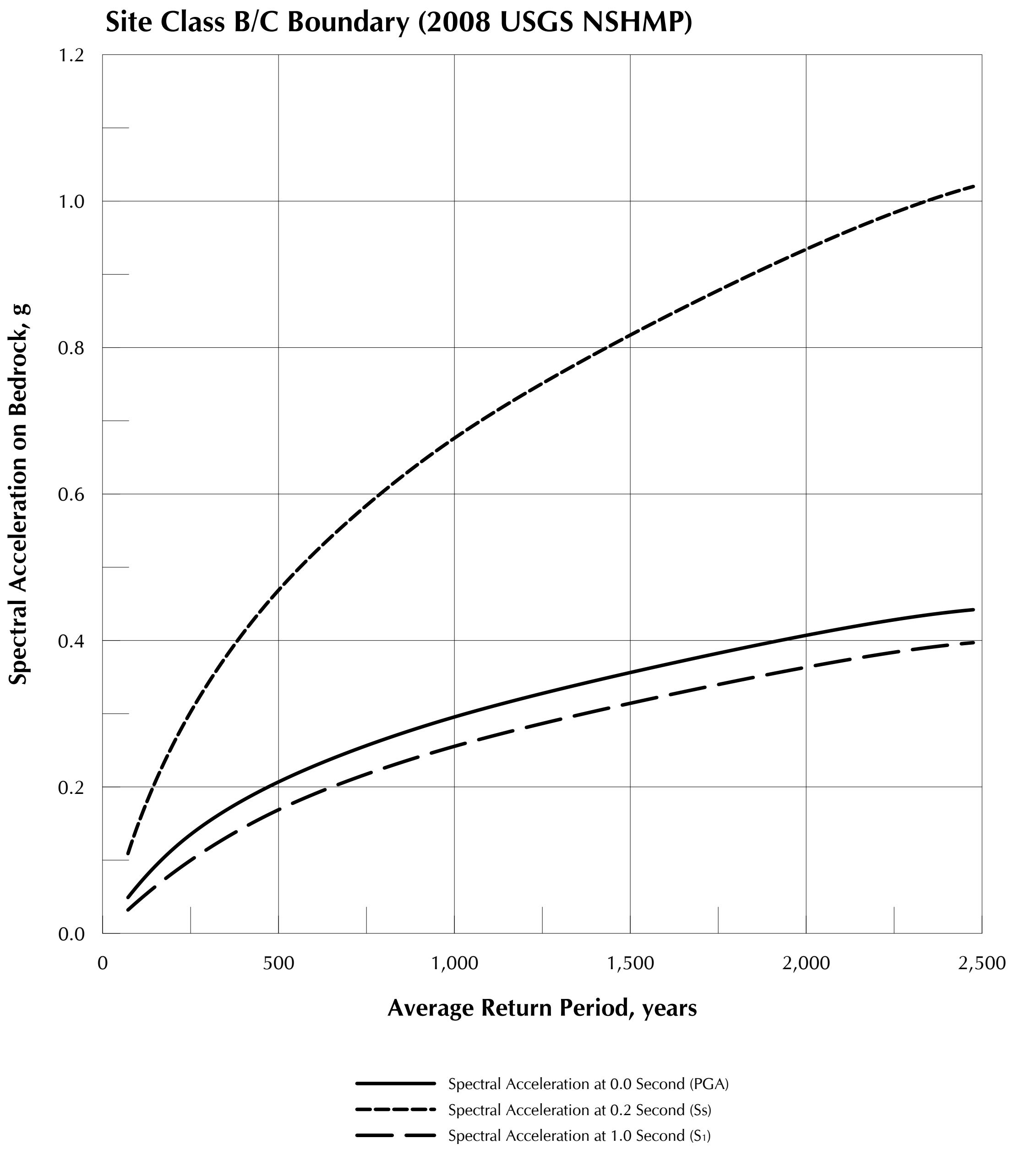

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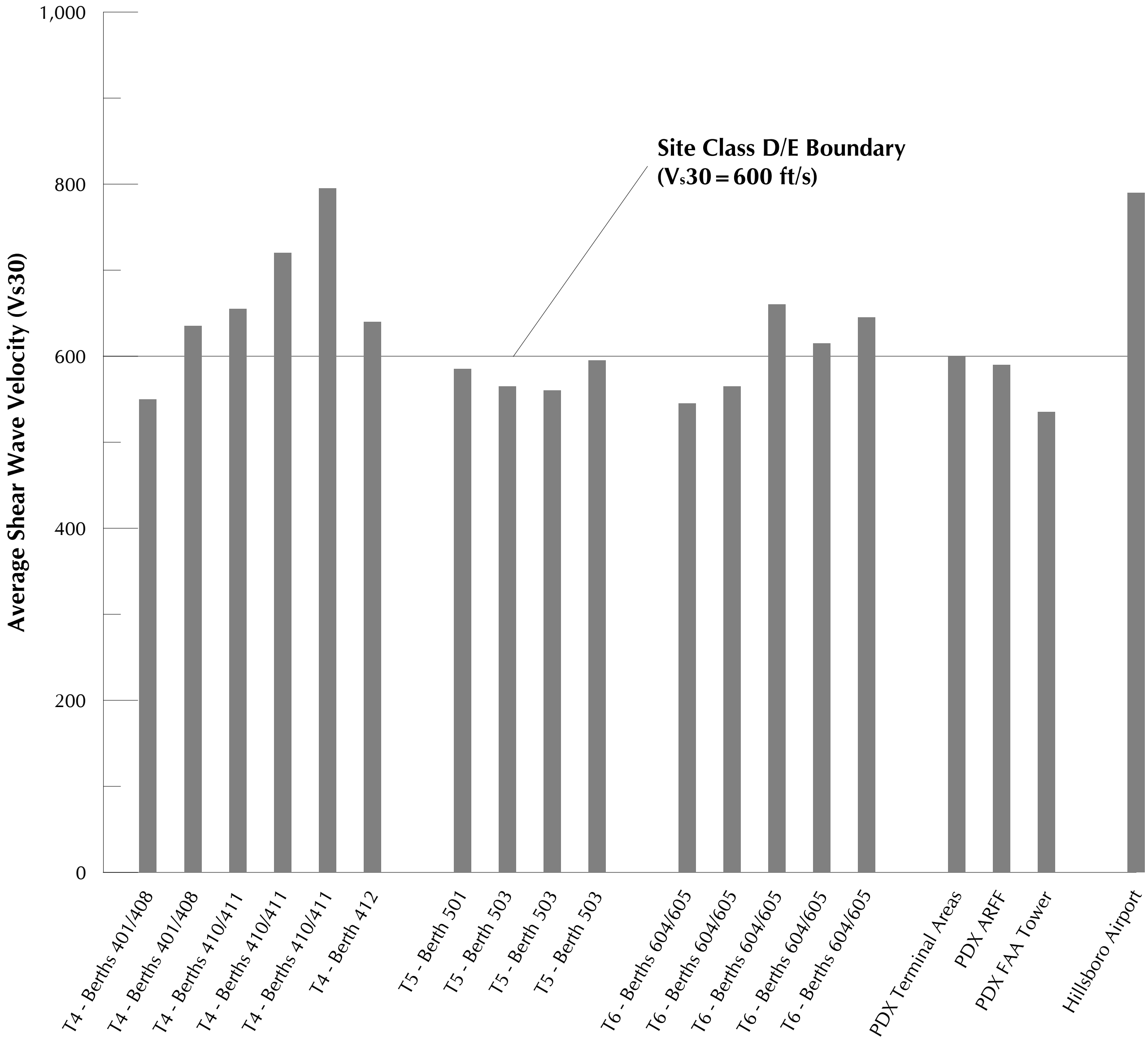
HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

SITE CLASS B/C BOUNDARY

SPECTRAL ACCELERATION ON BEDROCK
VS AVERAGE RETURN PERIOD

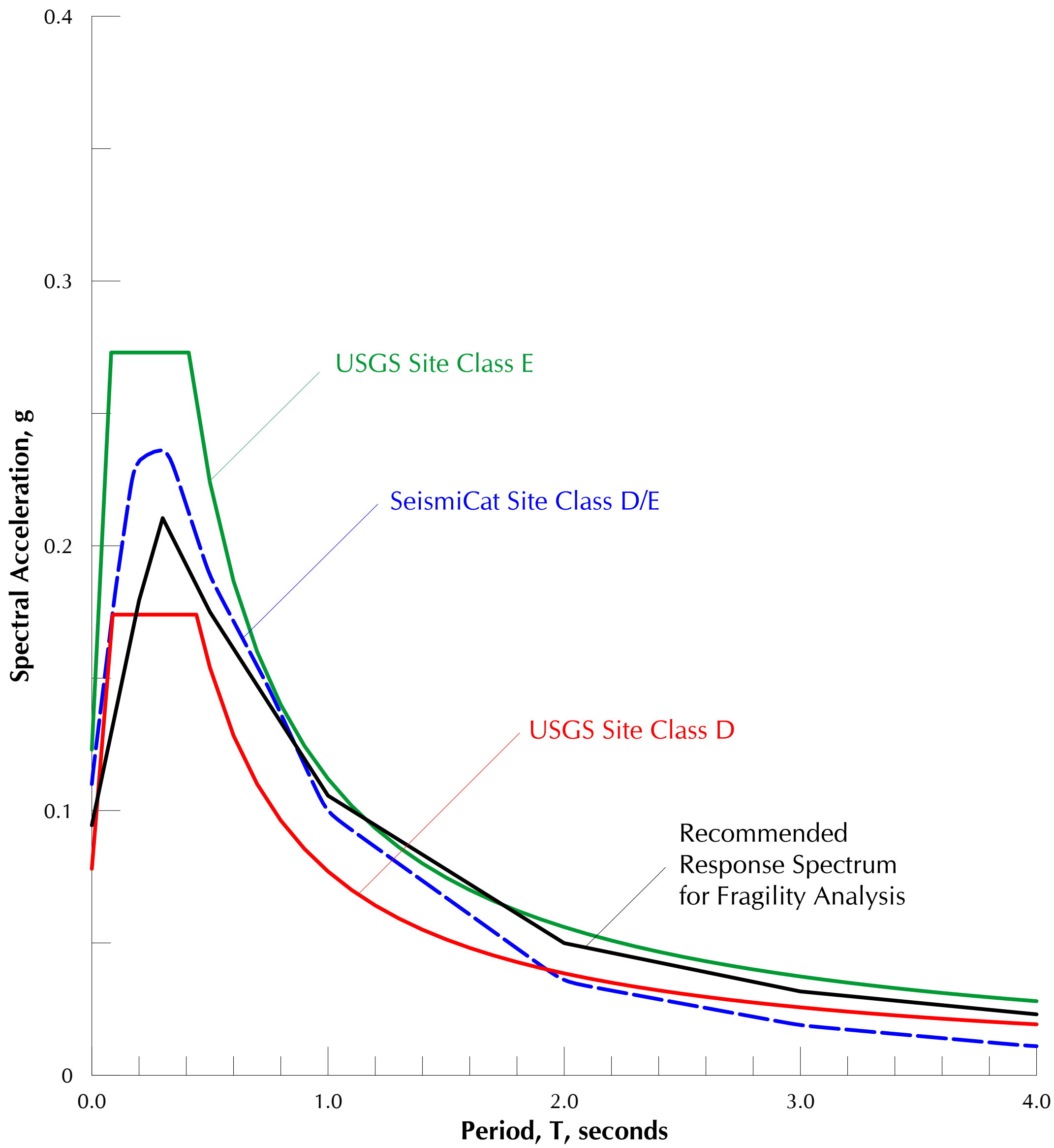
FEB. 2015 | JOB NO. 5550 | FIG. 2-1



GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

V_S30 HISTOGRAM

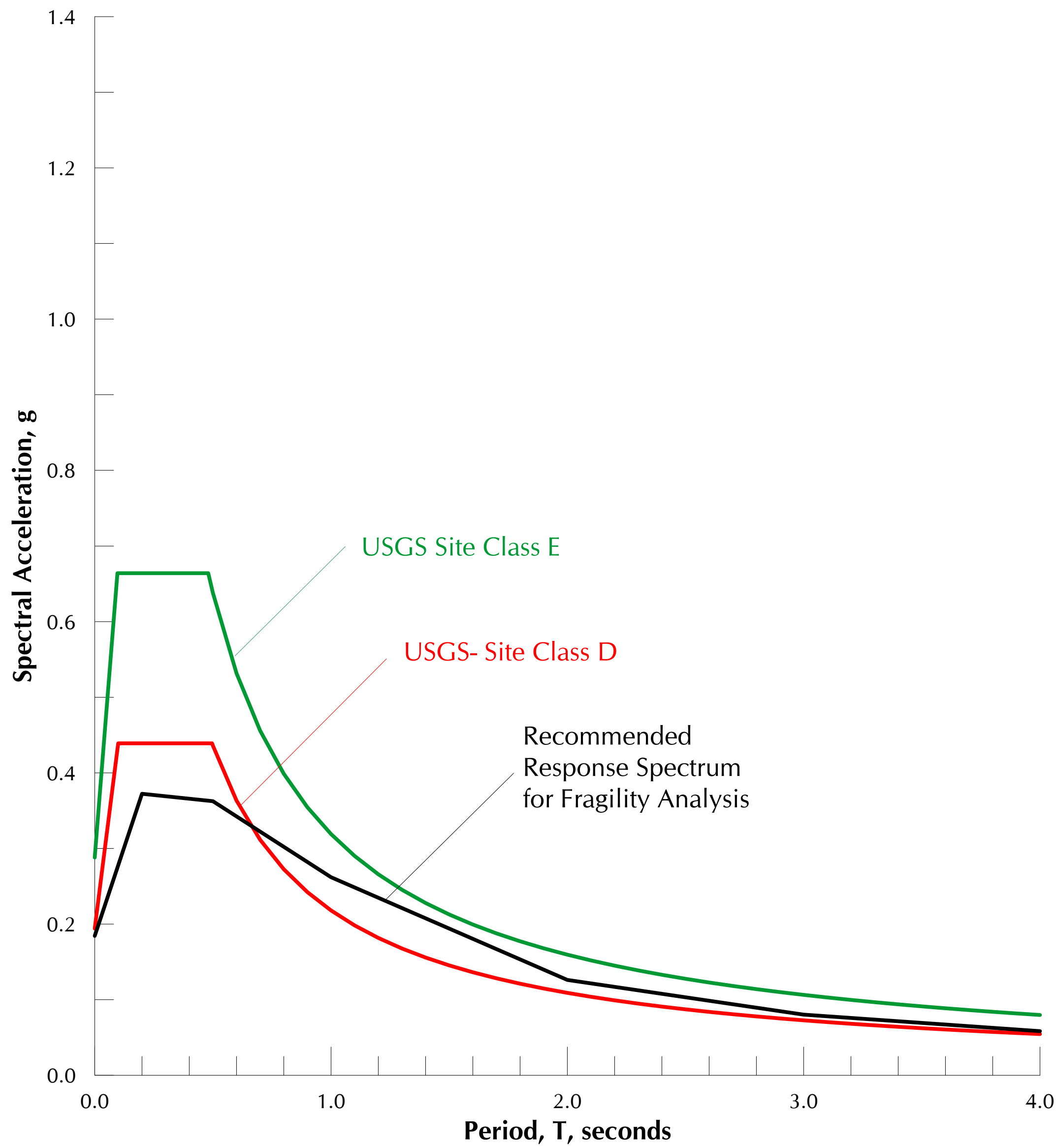


GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

72-YEAR HAZARD LEVEL SPECTRA
(5% DAMPING)

FEB. 2015 | JOB NO. 5550 | FIG. 2-3

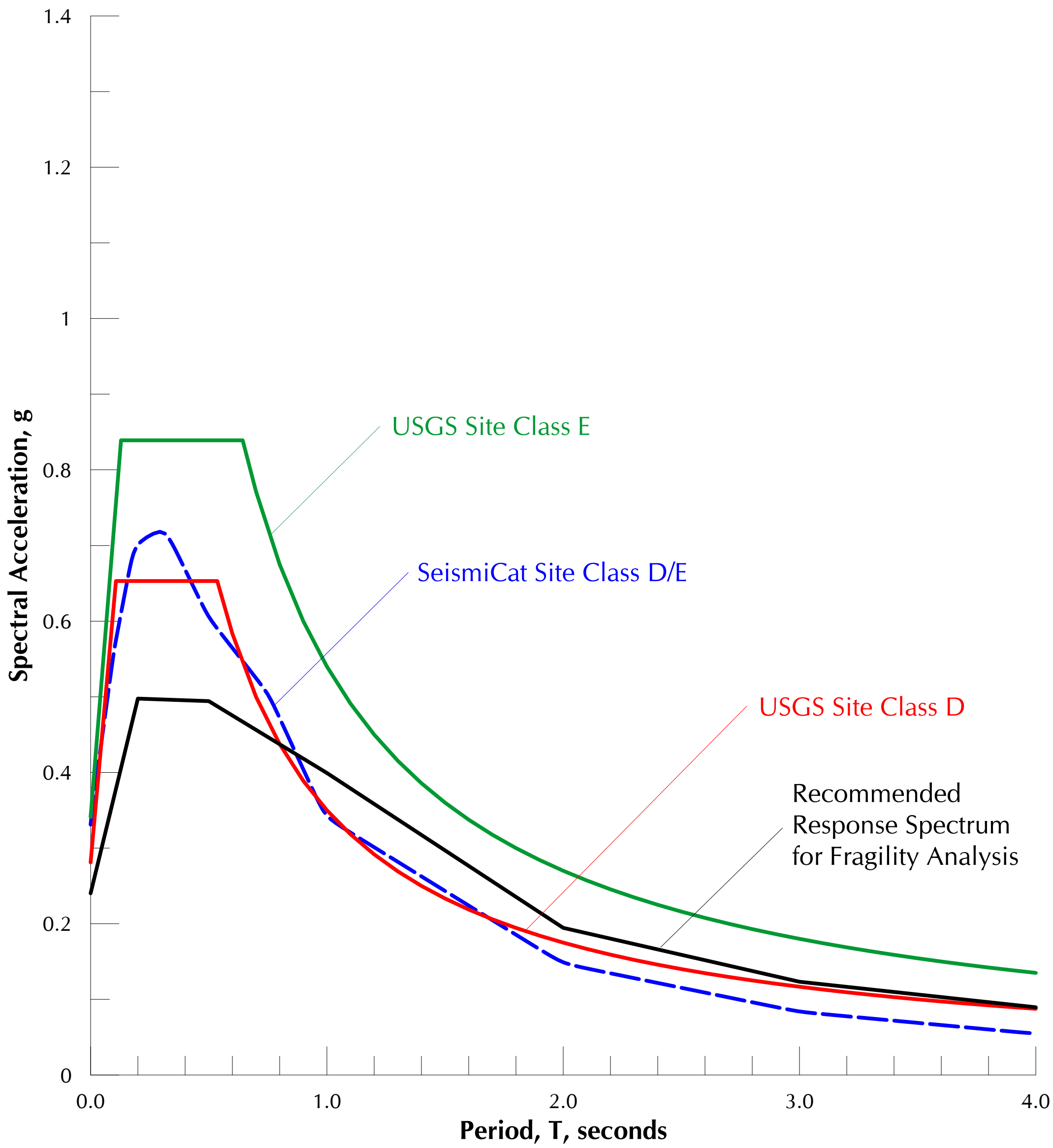


GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

224-YEAR HAZARD LEVEL SPECTRA
(5% DAMPING)

FEB. 2015 JOB NO. 5550 FIG. 2-4

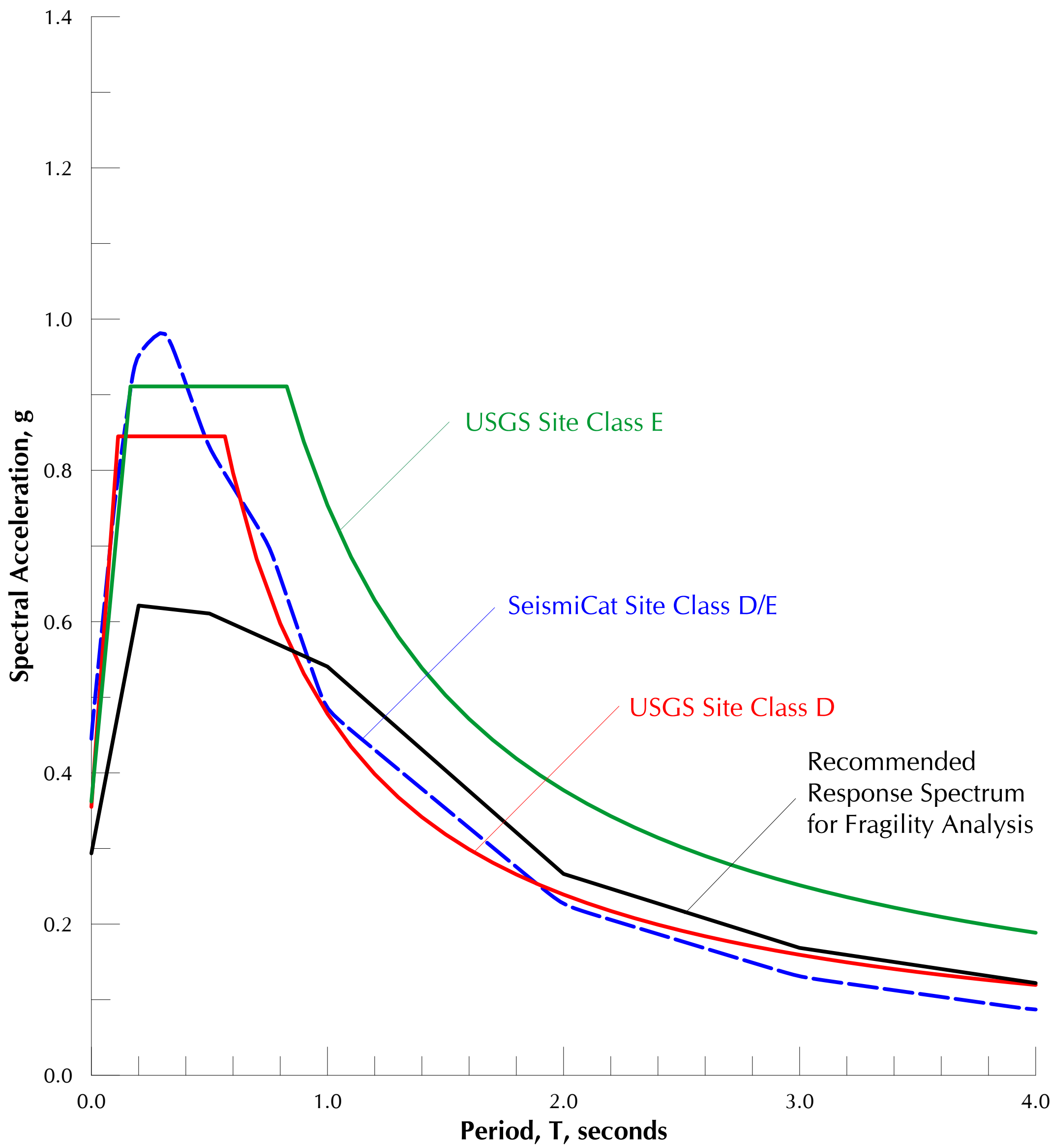


GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

475-YEAR HAZARD LEVEL SPECTRA
(5% DAMPING)

FEB. 2015 | JOB NO. 5550 | FIG. 2-5

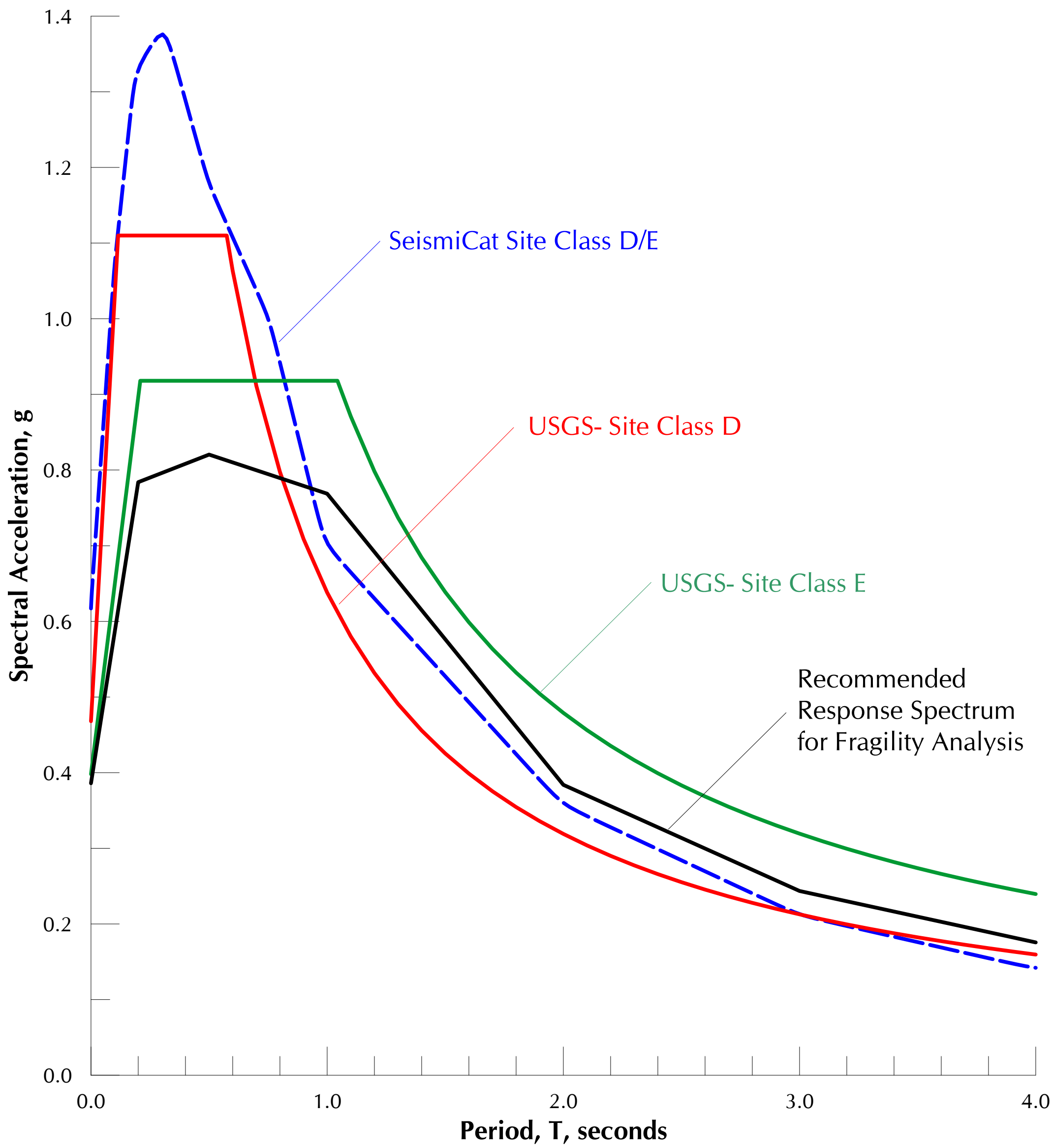


GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

975-YEAR HAZARD LEVEL SPECTRA
(5% DAMPING)

FEB. 2015 | JOB NO. 5550 | FIG. 2-6



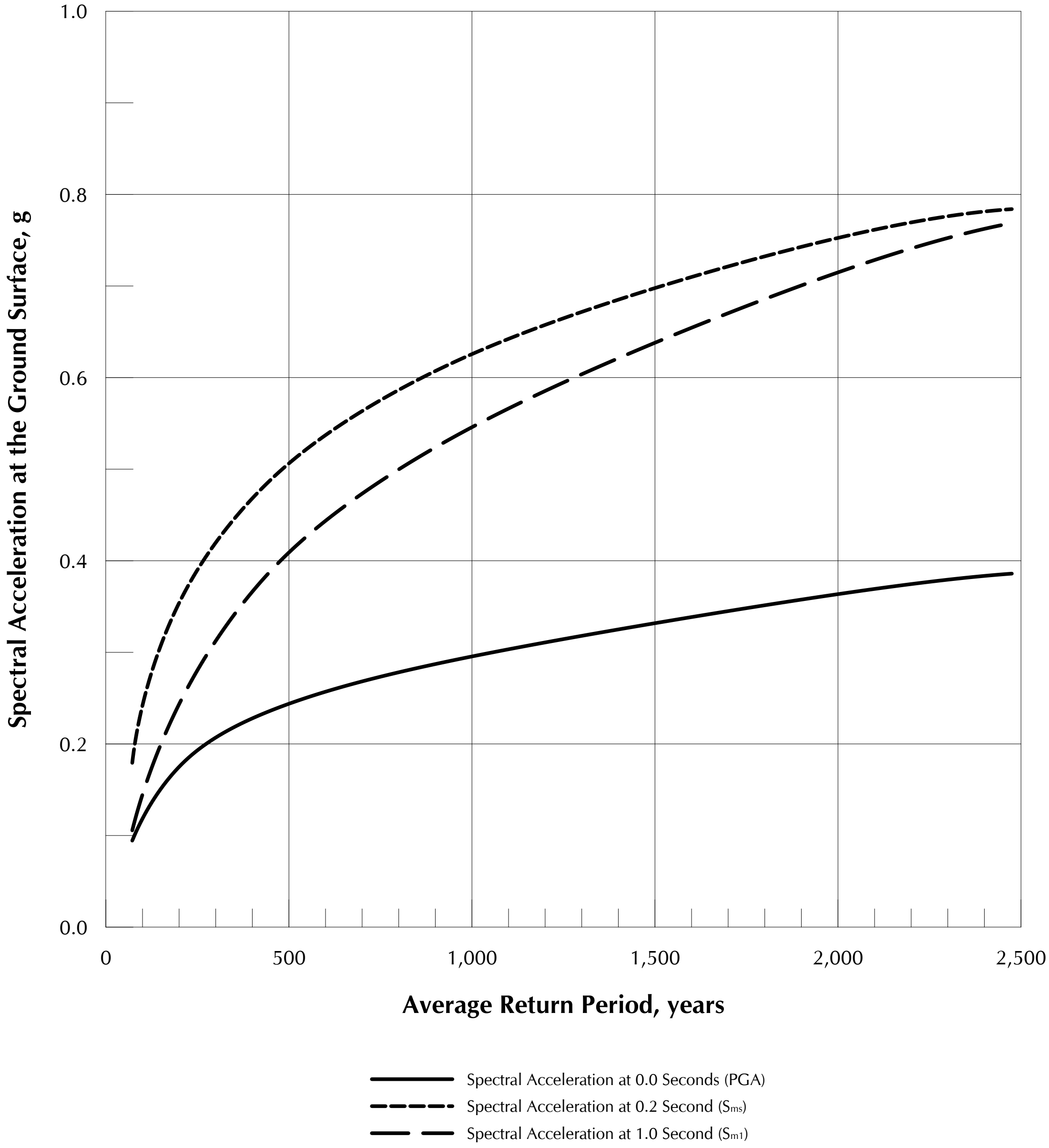
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

2,475-YEAR HAZARD LEVEL SPECTRA
(5% DAMPING)

FEB. 2015 JOB NO. 5550 FIG. 2-7

2008 USGS Site Class B/C and Port SAR Trends



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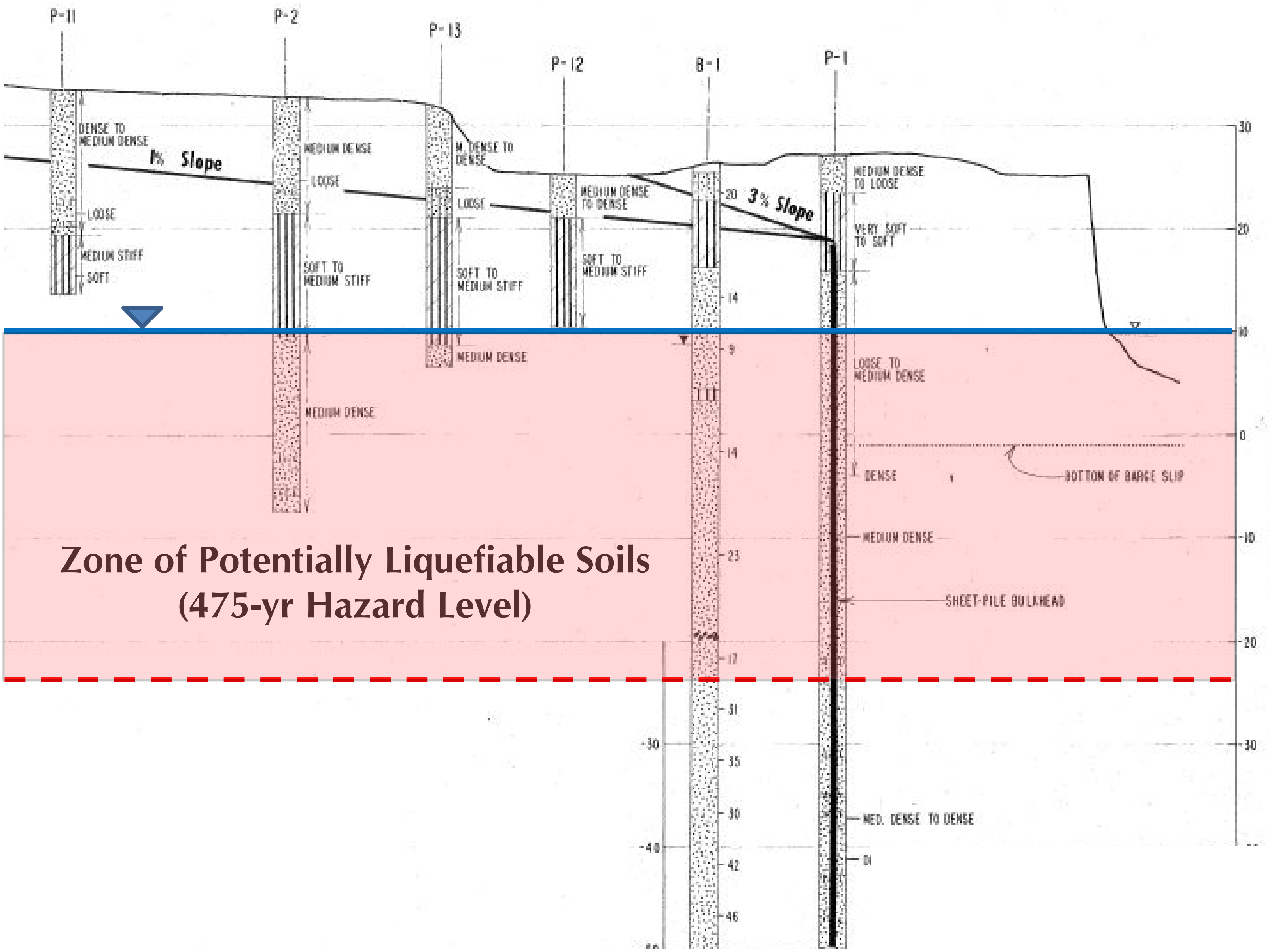
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

2008 USGS SITE CLASS B/C AND PORT SAR TRENDS

SPECTRAL ACCELERATION AT GROUND SURFACE
VS AVERAGE RETURN PERIOD

FEB. 2015 JOB NO. 5550 FIG. 2-8



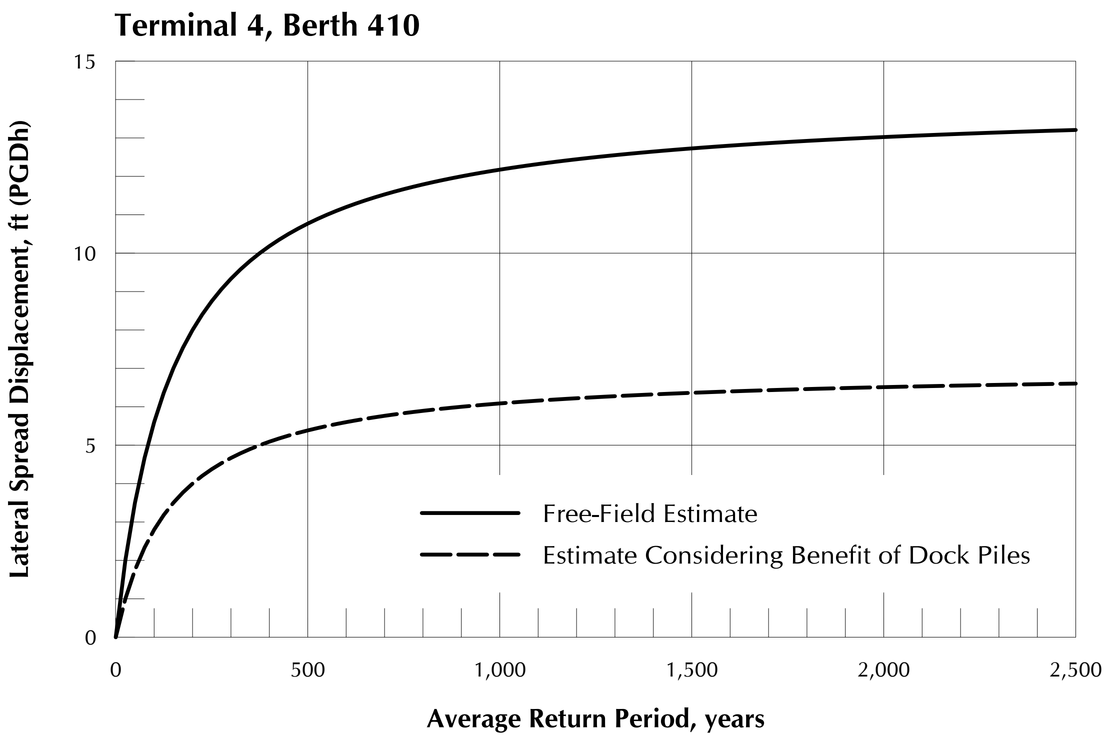
DRAWING FROM JUNE 7, 1985, REPORT BY GRI ENTITLED, "INTERIM REPORT, GEOTECHNICAL INVESTIGATION, TERMINAL T-6 MODULE FABRICATION SITE, RIVERGATE INDUSTRIAL PARK, PORTLAND, OREGON;" PREPARED FOR PORT OF PORTLAND

GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

EXAMPLE CROSS SECTION

FEB. 2015 JOB NO. 5550 FIG. 2-9



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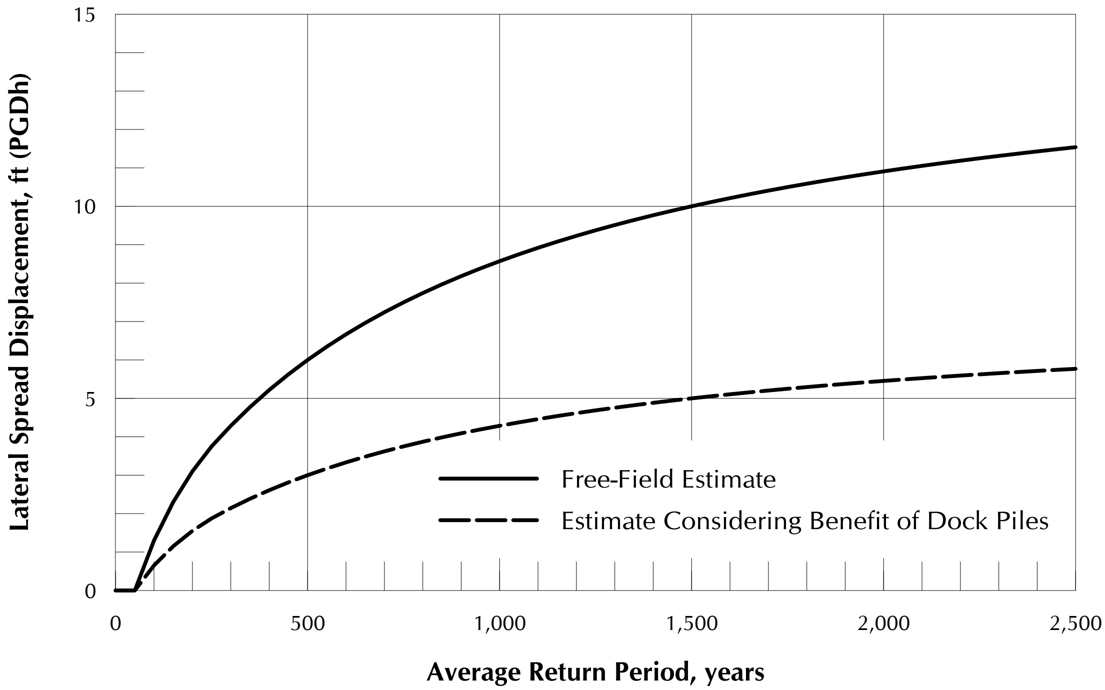
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 4, BERTH 410

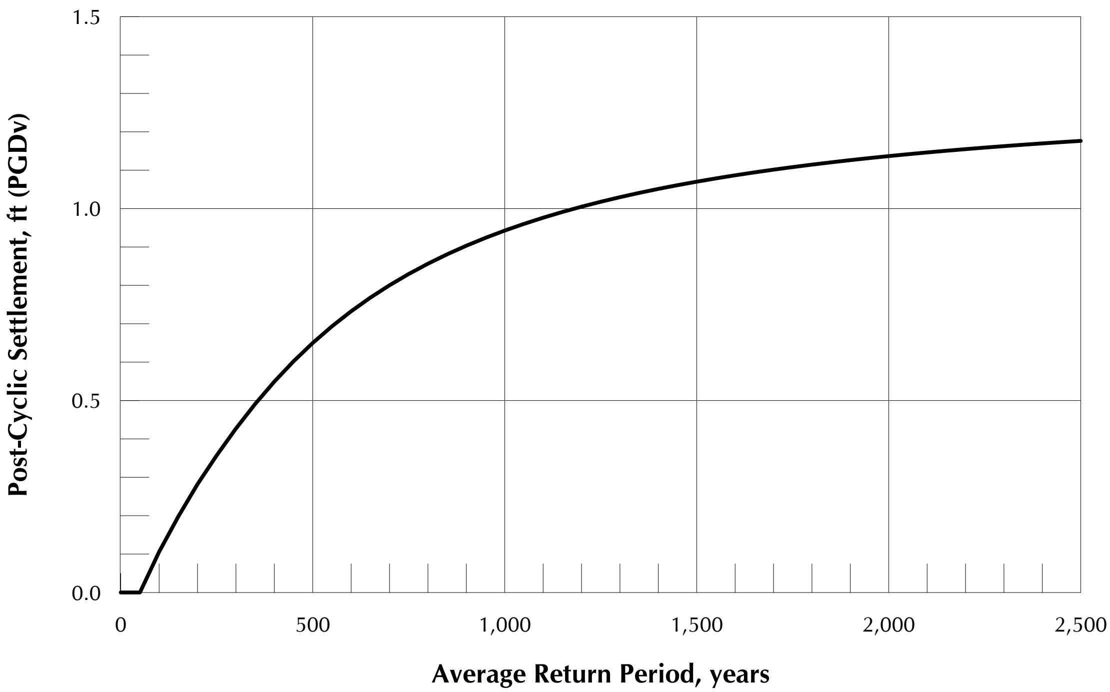
PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 JOB NO. 5550 FIG. 3-1

Terminal 4, Berth 411



Terminal 4, Berth 411

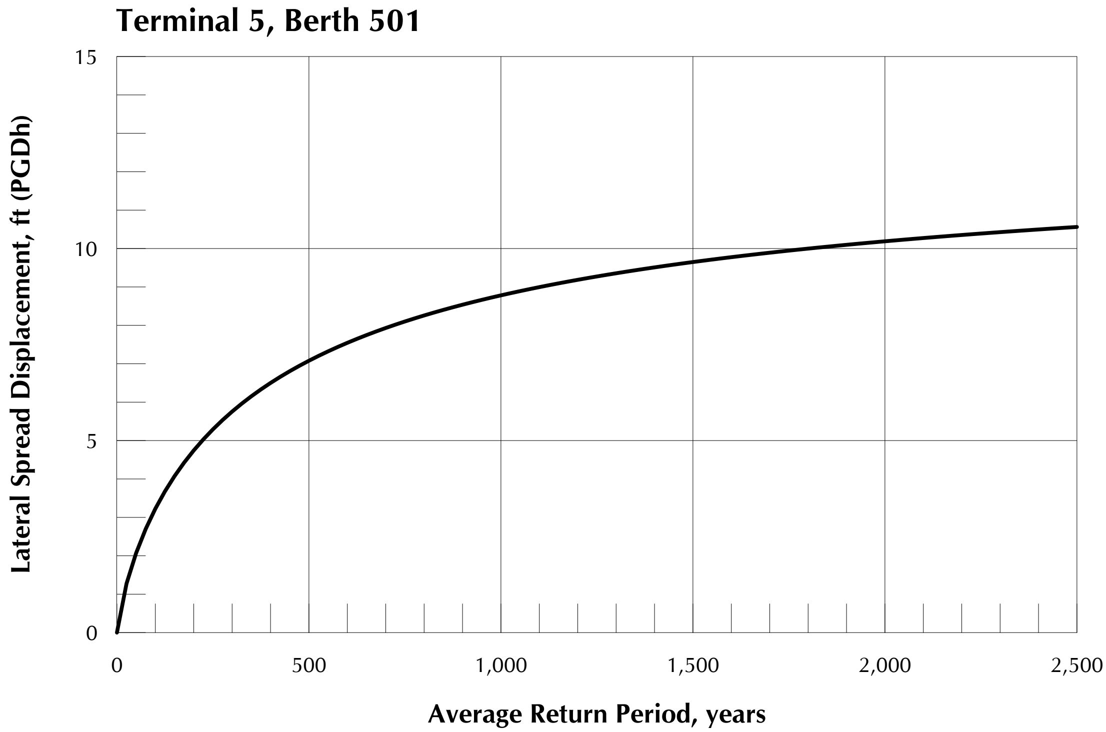


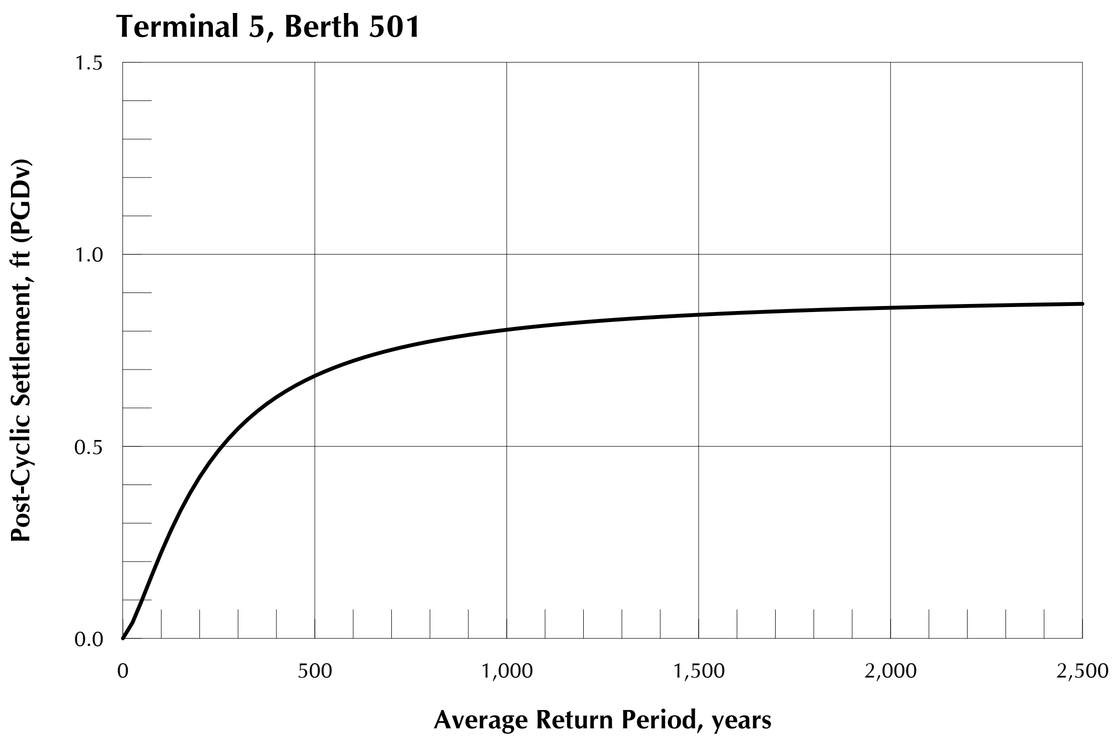
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 4, BERTH 411

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD





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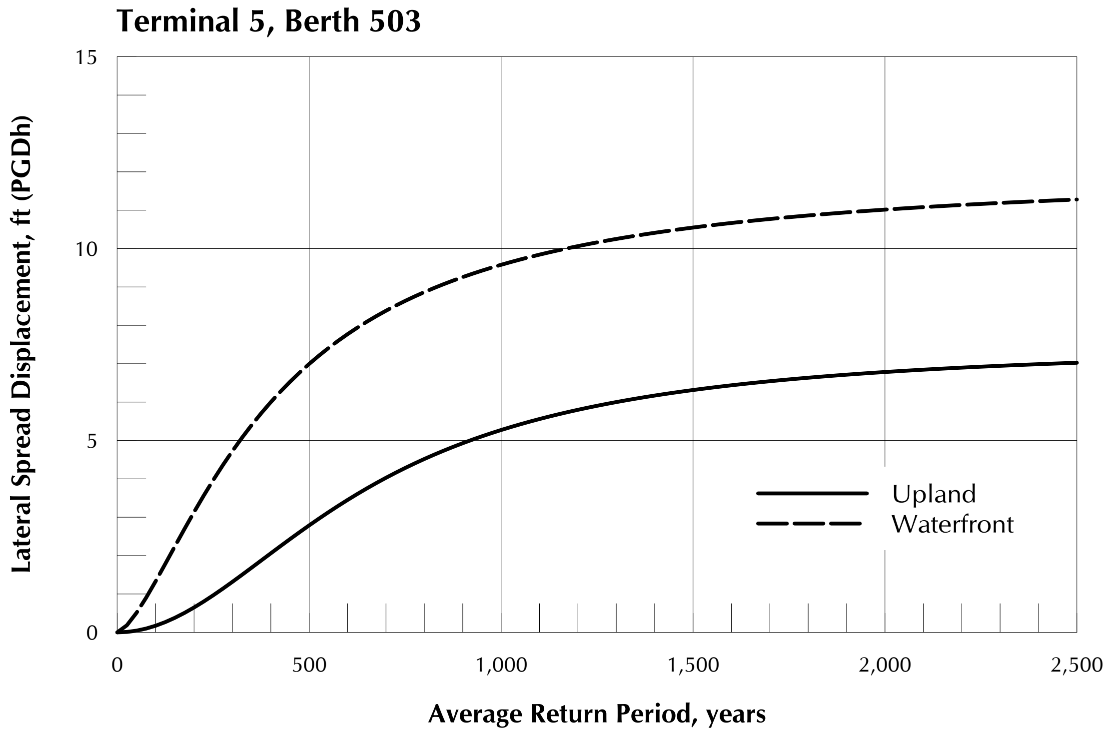
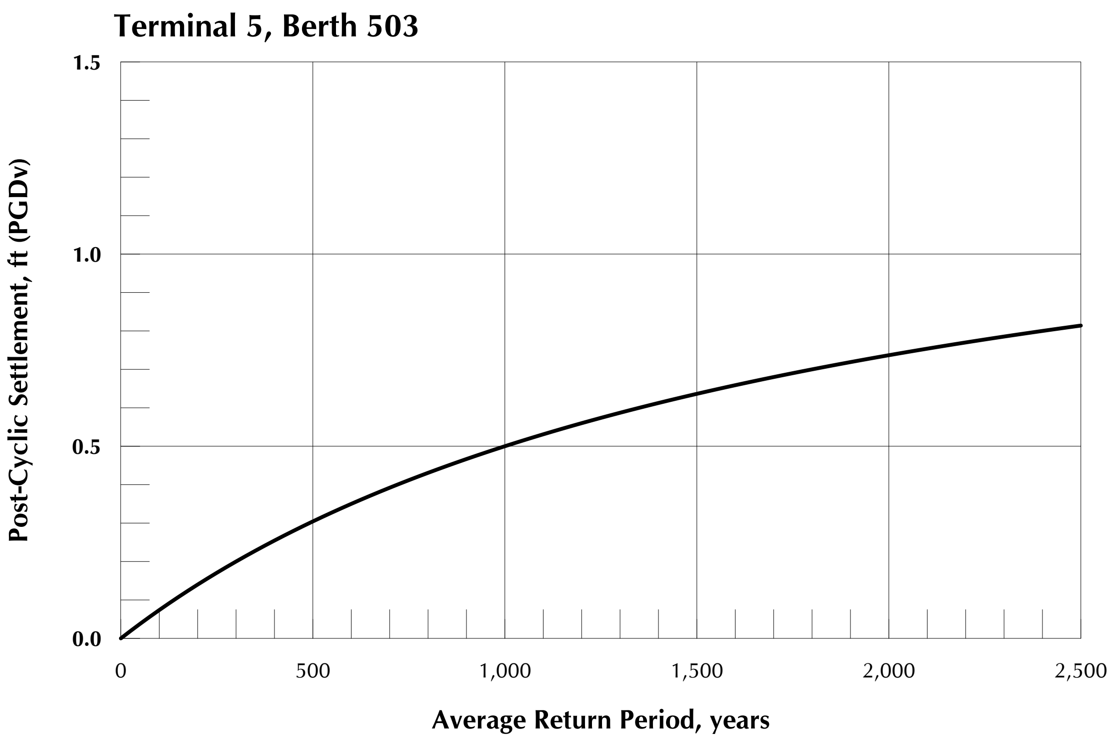
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 5, BERTH 501
PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-3





GRI

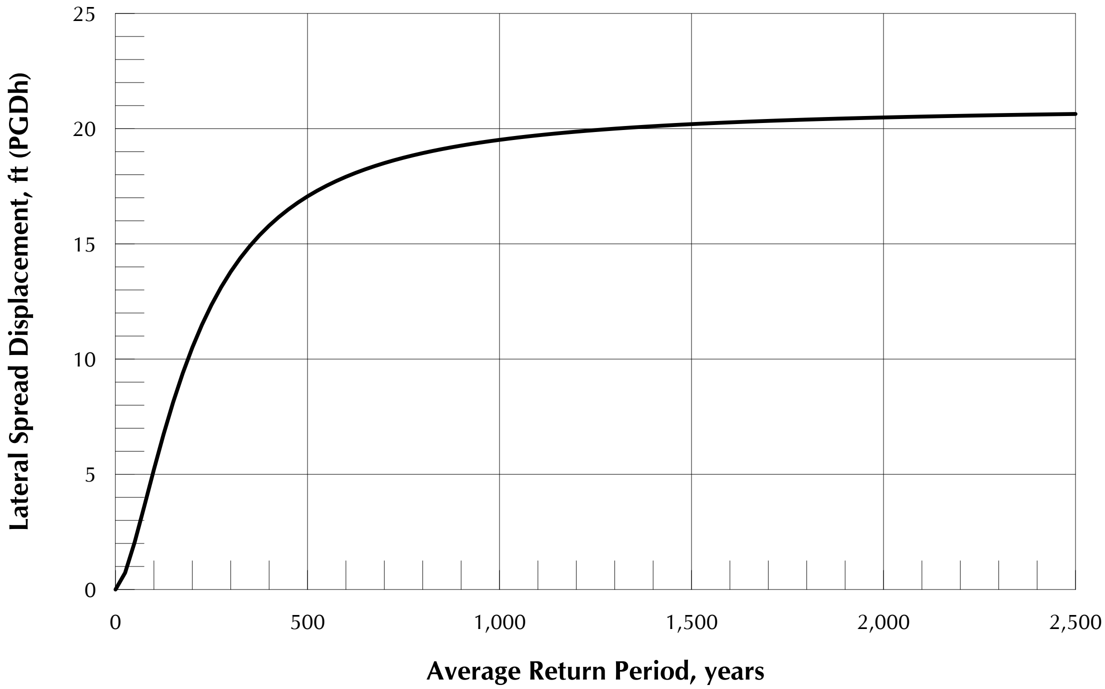
HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 5, BERTH 503

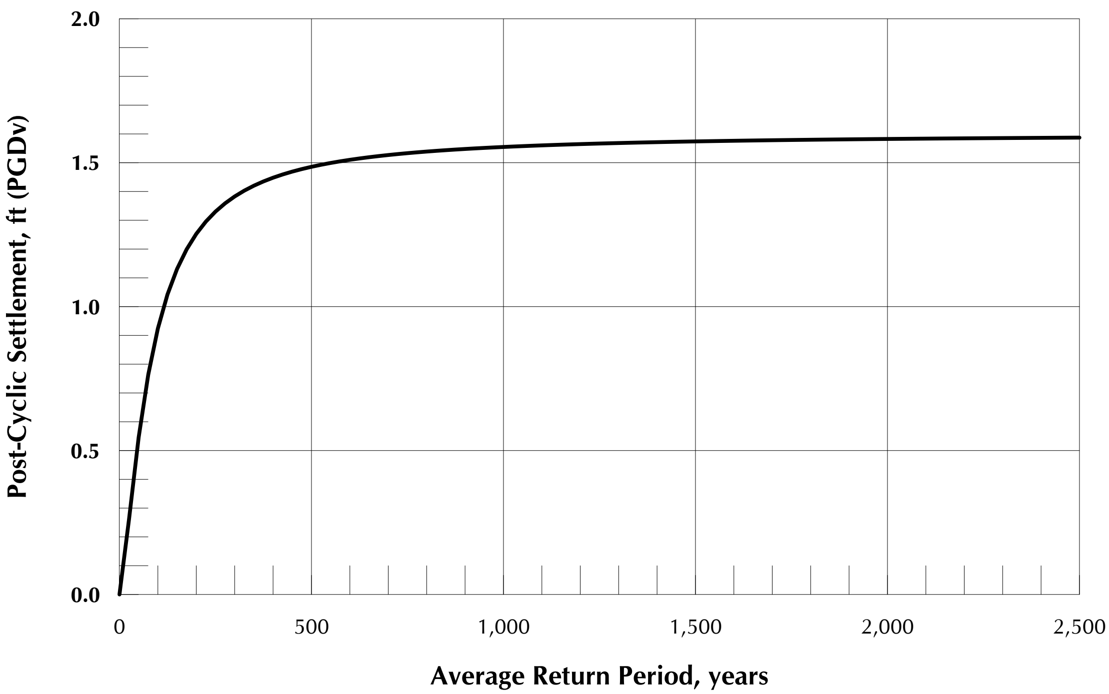
PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-4

Terminal 6, Berth 601



Terminal 6, Berth 601



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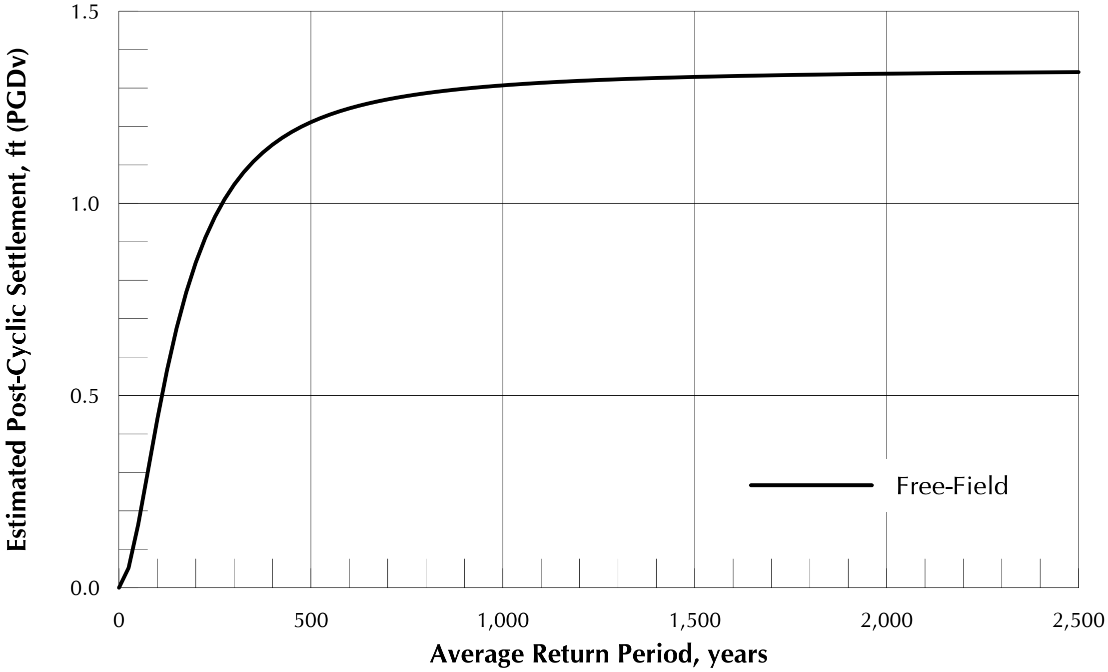
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 6, BERTH 601

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

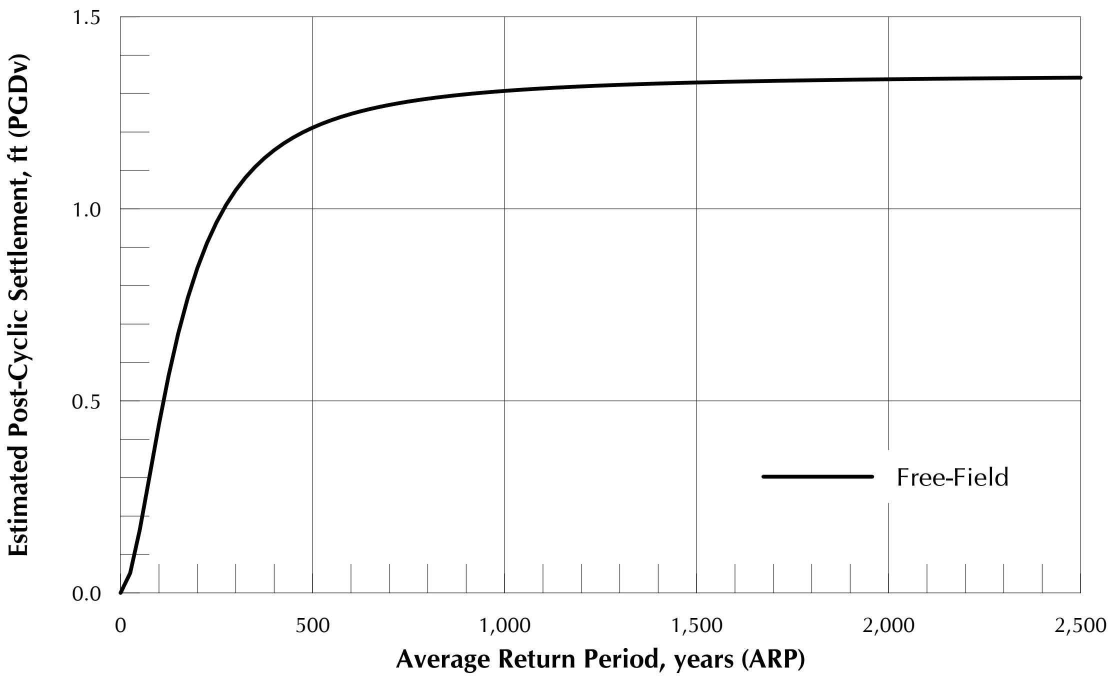
FEB. 2015 JOB NO. 5550 FIG. 3-5

Terminal 6, CDC Warehouse



Note: Additional settlement of continuous perimeter footings is not anticipated. Additional settlement of isolated column footings is not anticipated at the 72-yr hazard level. To estimate the additional settlement of isolated column footings at return periods greater than 72 yrs, add 0.5 in. to the free-field settlement.

Terminal 6, Electrical Shop



Note: Additional foundation settlement is not anticipated at the 72-yr hazard level. To estimate building settlement at return periods greater than 72 yrs, add 0.5 in. to the free-field settlement.

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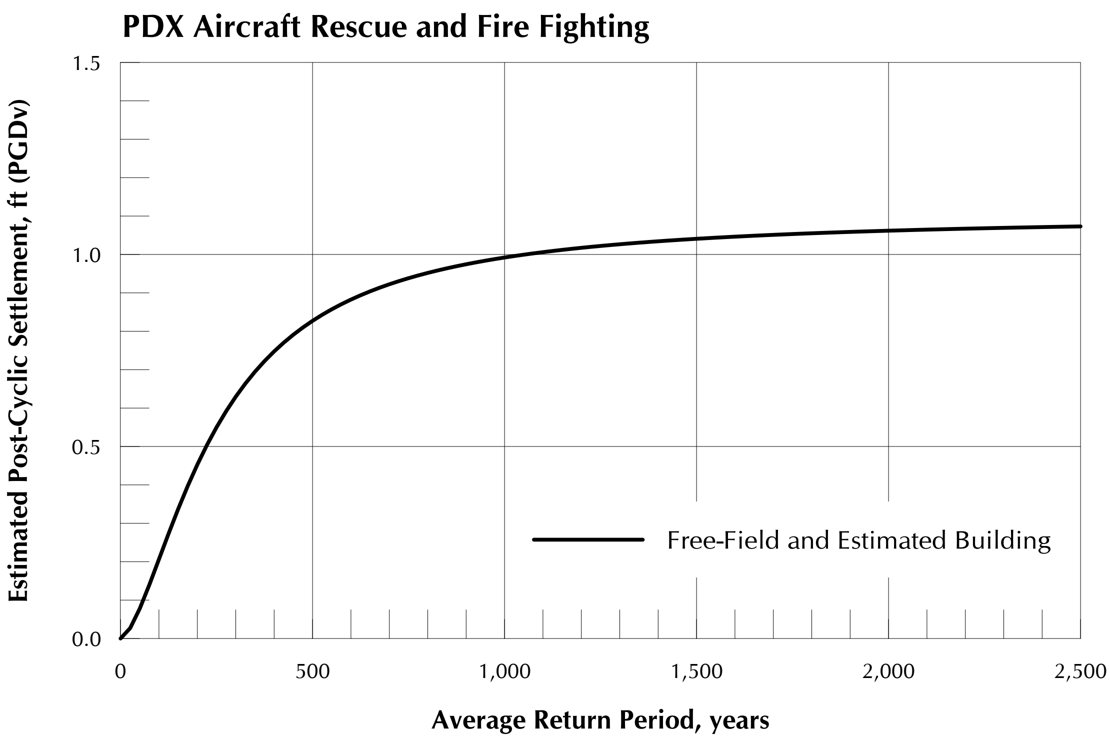
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

TERMINAL 6, CDC WAREHOUSE AND ELECTRICAL SHOP

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 JOB NO. 5550 FIG. 3-6



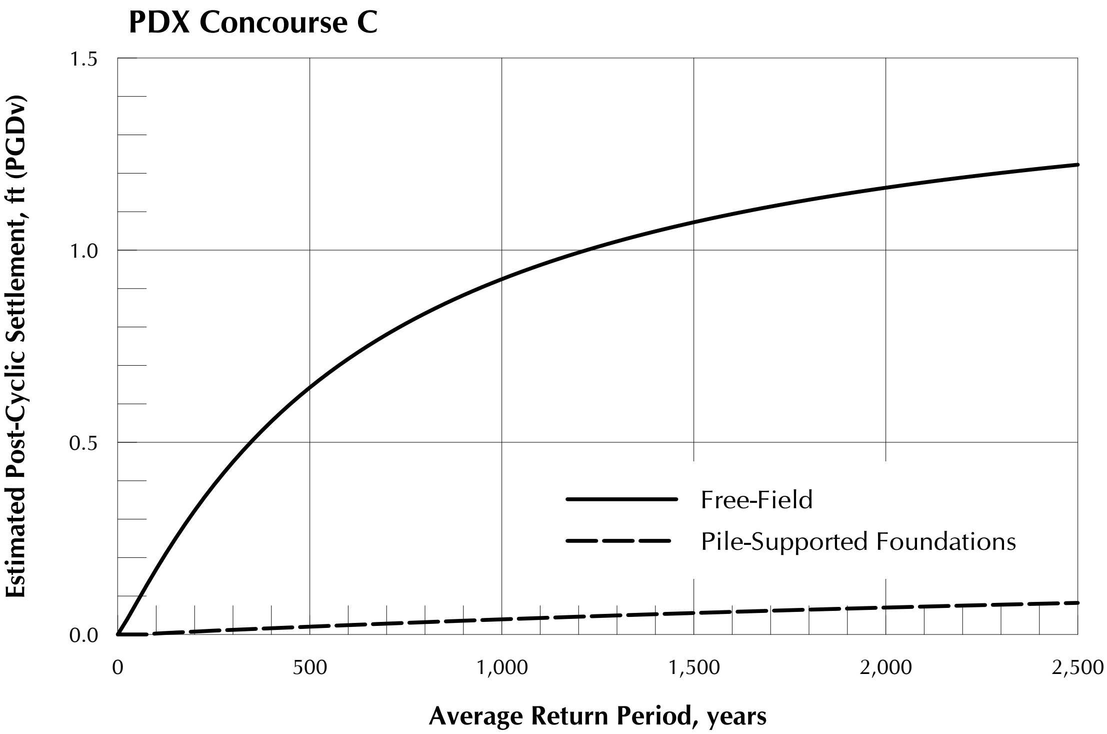
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX AIRCRAFT RESCUE AND FIRE FIGHTING

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-7



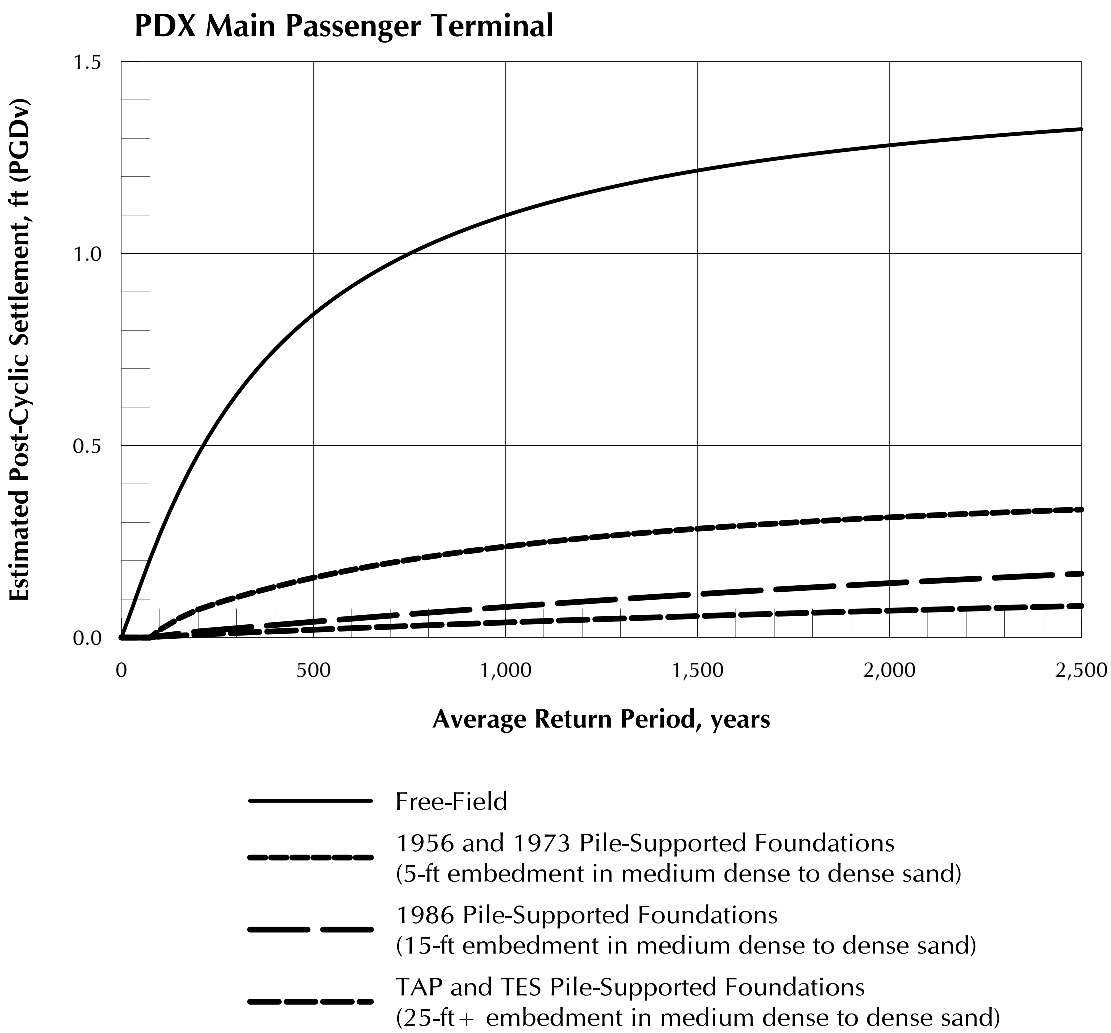
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX CONCOURSE C

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-8



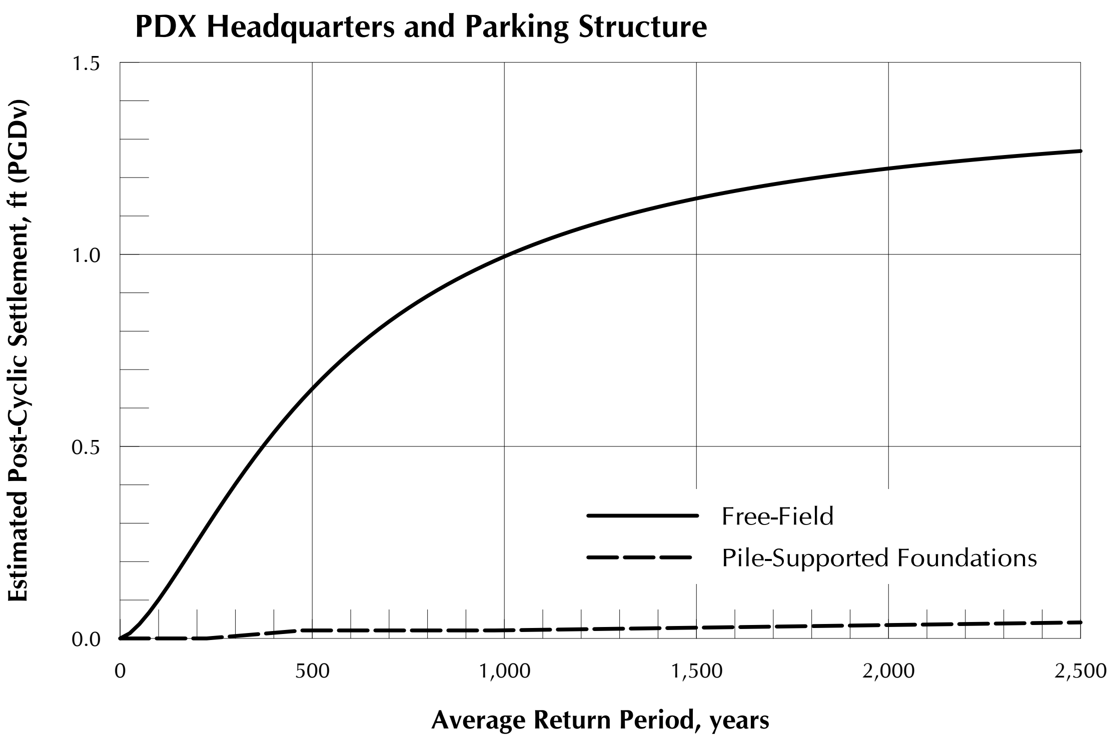
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX MAIN PASSENGER TERMINAL

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-9



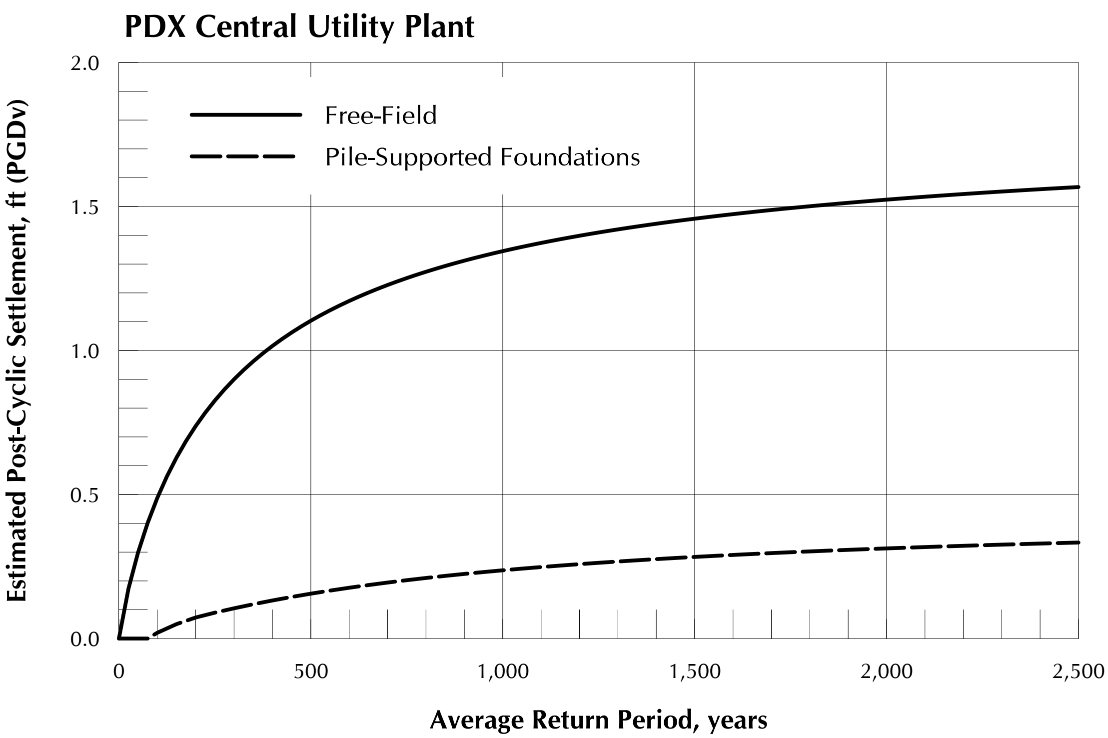
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX HEADQUARTERS AND PARKING STRUCTURE

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 JOB NO. 5550 FIG. 3-10

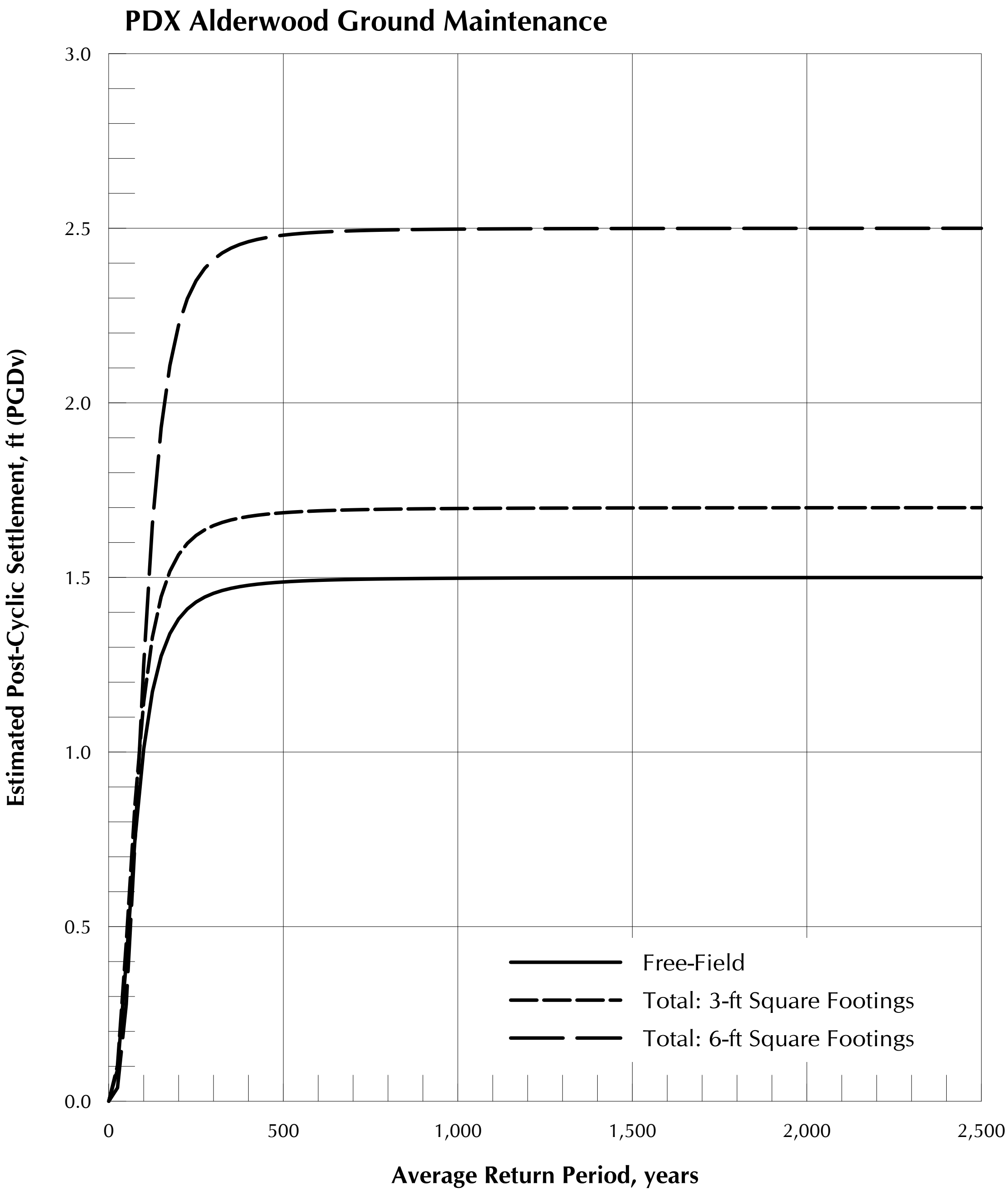


GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX CENTRAL UTILITY PLANT
PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 JOB NO. 5550 FIG. 3-11



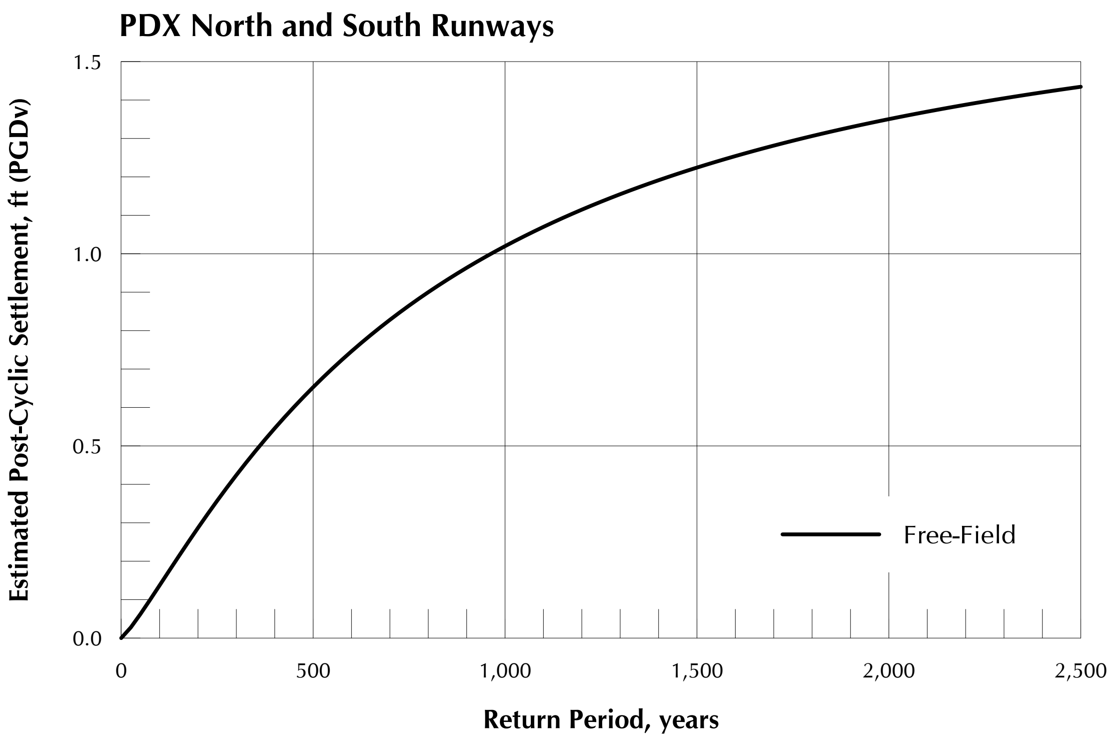
GRI

HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX ALDERWOOD GROUND MAINTENANCE FACILITY

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-12



HNTB CORPORATION
PORT OF PORTLAND SEISMIC RISK ASSESSMENT

PDX NORTH AND SOUTH RUNWAYS

PERMANENT GROUND DEFORMATION
VS AVERAGE RETURN PERIOD

FEB. 2015 | JOB NO. 5550 | FIG. 3-13

Appendix 3 – Seismic Assessment of Building Assets

(kpff Consulting Engineers)

kpff

Port of Portland
Seismic Risk Assessment

Building Assets

February 2015

KPFF Consulting Engineers
111 SW Fifth Avenue, Suite 2500
Portland, OR 97204-3628

Table of Contents

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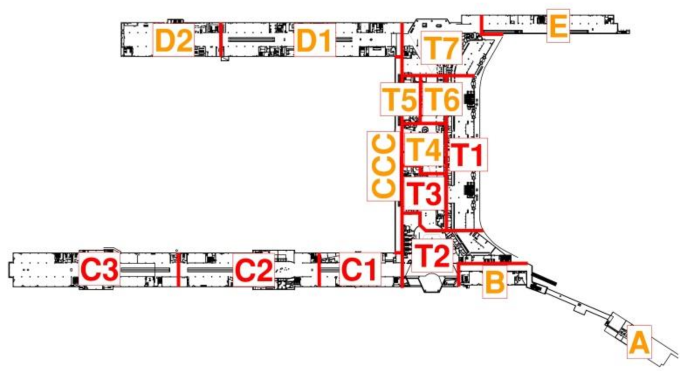
Building Assets Summary

KPFF reviewed all the buildings listed below, identified by the Port as key assets, for their expected performance in a Code level seismic event.

Next, KPFF further reviewed the assets identified below as Critical BI Assets, that are considered critical relative to their effect on business interruption (BI) of the Port's functions. This included assessment of the expected building performance relative to different performance objectives, as well as possible mitigation strategies and costs, to achieve Immediate Occupancy performance if subjected to a 475-year return period earthquake.

Critical BI Assets:

- 1 - Central Utility Plant (CUP)
- 3 - Concourse C
    - 3 C1 - Concourse C East
    - 3 C1 - Concourse C Central
    - 3 C1 - Concourse C West
- 4 - Terminal, building sections:
    - 4 T1 - Ticket Lobby
    - 4 T2 - South Node
    - 4 T3 - Oregon Market Place – South



Other Key Assets:

- 4 - Terminal, building section:
    - 4 T4 - Oregon Market Place – Central
- 5 - Aircraft Rescue and Firefighting Facility (ARFF)
- 6 - Port Headquarters and Parking, building sections
    - 6a - HQ/P2 (North)
    - 6b - P2 (South)
- 13 - Ground Maintenance Administration and Shops (Building B)
- 14 - Ground Maintenance Facility (Building A)
- 15 - Ground Maintenance Facility (Building C)
- 16 - Maintenance Warehouse at T-6
- 17 - Electrical Shop at T-6

Summary of Findings

The review of the buildings listed above included a site walk and review of existing drawings, along with limited analysis, assessing the expected building performance considering the site-specific geotechnical information on liquefaction and seismic response spectra provided by GRI and New Albion as part of this study. Findings were input and coordinated with the SeismiCat portfolio analysis.

The buildings have been constructed with different materials and under different codes over a span of almost 60 years. Some buildings have undergone renovations, some more than once. Consequently, thes assets suffer from seismic deficiencies to varying degrees for factors listed below and summarized qualitatively in Figure 1. A more detailed review of each asset and possible mitigation strategies for critical BI assets are described in sections following. The Supplement contain definitions, mitigation cost estimates, a summary of the original design Code seismic parameters, summary of building foundations, and a description of Code performance levels and methodology with a progression of the building Code and design parameters in Oregon.

- Liquefaction – Most of the buildings have slab-on-grade base levels, and will suffer effects from soil settlement. This will impact not only the slab itself, but any MEP, baggage, or other systems supported by the slab. Older piles do not appear to penetrate the denser sand layers adequately and may experience settlements of several inches, which will be nonuniform from column to column. This differential settlement will distress the structure and any rigid non-structural elements. Buildings with spread footings will experience severe deformations from settlements.
- Capacity – Older structures that have not been seismically upgraded are deficient in strength to resist the current Code prescribed seismic forces. Newer structures, or those that have been upgraded, generally have adequate strength for the Code level performance objective; however, may be deficient for the higher performance objective of Immediate Occupancy at a 475-year return period event.
- Ductility – Many of the structures are deficient with respect to current Code requirements for ductility and have systems that are either no longer permitted for new construction in this seismic region, and/or are penalized in more recent codes for their lack of ductility by requirements to design for significantly greater loads. Ductility includes concrete shear wall boundaries, brace connections, braced frame columns, drag connections, diaphragm connections, and similar items.

Figure 2 indicates the performance of the critical BI assets relative to the desired objective of Immediate Occupancy at the 475-year return period earthquake. Also indicated is the approximate return period earthquake for which the existing building would be expected to perform to Immediate Occupancy, Life Safety, or Collapse Prevention, considering ASCE 41 design methodology along with the site-specific response spectra developed by GRI and New Albion engineers, excluding the effects of liquefaction. <u>It is important to stress that Immediate Occupancy objectives will not be achieved in buildings experiencing significant liquefaction effects.</u> As further discussed in the geotechnical sections, these effects are expected to occur at relatively low level earthquakes. Additionally, Immediate Occupancy will not be fully achieved without addressing potential deficiencies in support, bracing, tie-ins, and jointing of secondary structural elements. This includes MEP equipment and systems, exterior skin system, glazing, ceilings, and similar elements. The Code drift limitations for Immediate Occupancy are more stringent in order to mitigate effects of damage to the building shell as well as reducing damage to interior elements. These elements, connections and joints should be reviewed in detail for capability to achieve the desired performance.

Existing Building Condition - OSSC Code

Building	Year Built	Seismic Upgrade ?	Liquefaction		LFRS	
			Foun-dations	Slab-on-grade	Force Capacity	Ductility
1 - CUP	’72, ’92, ’94	1998	Short Piles			
3 - Concourse C	1999	NO	Long Piles			
4 T1 - Terminal Ticket Lobby	’73, ‘96	1996	Mixed Piles			
4 T2 - Terminal South Node	1999	NO	Long Piles			
4 T3 - Terminal OMP South	’56, ’86, ’02	2002	Mixed Piles			
4 T4 - Terminal OMP Central	1956	2006* (partial)	Mixed Piles			
5 - ARFF	1996	NO	Mat			
6a - HQ/P2 (North)	2007	NO	Long Piles			
6b - P2 (South)	2007	NO	Long Piles			
13 - Ground Maint. Facility B	1982	NO	Spread			
14 - Ground Maint. Facility A	1982	NO	Spread			
15 - Ground Maint. Facility C	1982	NO	Spread			
16 - Maintenance Warehouse	1973	NO	Spread			
17 - Electrical Shop	1982	NO	Spread			

Notes:

1. This table is based on the original design objective of Standard, Special or Essential Facility occupancy for each asset.
2. * Without completion of the voluntary seismic upgrade.

Legend:

Good
Fair
Poor

Figure 1

kpff

Existing Building I.O. Performance Summary

(475-year Immediate Occupancy for Critical BI Assets)

Building	Current I.O. Quake* (years)	Current L.S. Quake* (years)	Liquefaction Effects		LFRS	
			Foun-dations	Slab-on-grade	Force Capacity	Ductility
1 - CUP	110	270	Short Piles			
3 - Concourse C	130	1000	Long Piles			
4 T1 - Terminal Ticket Lobby	600	920	Mixed Piles			
4 T2 - Terminal South Node	1700	2500	Long Piles			
4 T3 - Terminal OMP South	140	1400	Mixed Piles			

Notes:

1. This table is based on the unmitigated condition of the building for the design objective of Immediate Occupancy, considering the site specific seismic response spectra for the 475-year event.
2. * Condition *without* consideration of liquefaction effects. Immediate Occupancy will not be achieved with significant liquefaction effects.
3. I.O. = Immediate Occupancy Performance Level.
4. L.S. = Life-Safety Performance Level.

Legend:

Good
Fair
Poor

Figure 2

Figure 3 includes the estimated downtimes for building assets. These downtimes are based on engineering judgment of the time to repair/rebuild the structure to an occupiable condition after a 475-year event, considering the type of structure, expected resilience, and effects of liquefaction. These estimates are intended to be conservative estimates, so as not to provide an artificially high B/C ratio in the cost analyses performed by ImageCat. They do not include potential effects on downtime of procuring funding or permits, availability of contractors, construction supplies, or design consultants, nor disruption to utilities outside of the building, vehicular access to PDX, Port communications and personnel issues, and similar concerns.

Estimated Building Downtime Summary
(for 475-year earthquake)

Building	Estimated Downtime (months)
1 - CUP	12
3 - Concourse C	2
4 T1 - Terminal Ticket Lobby	12
4 T2 - Terminal South Node	2
4 T3 - Terminal OMP South	24
4 T4 - Terminal OMP Central*	24
5 - ARFF	2
6a - HQ/P2 (North)	1
6b - P2 (South)	1
13 - Ground Maint. Facility B	16
14 - Ground Maint. Facility A	16
15 - Ground Maint. Facility C	16
16 - Maintenance Warehouse	12
17 - Electrical Shop	12

* Considering completion of the remaining voluntary seismic upgrade elements

Figure 3

Mitigation for the critical BI assets would address the general deficiencies in the primary structural systems noted in Figure 1 above, along with improving foundations and slab-on-grade for liquefaction, as well as addressing the architectural and MEP systems for bracing and accommodation of building drift. The intent of the mitigation schemes is to achieve as close as practical to Immediate Occupancy performance. Rough order of magnitude construction cost estimates for the mitigation projects are shown in Figure 4 below. These include estimated direct and indirect costs, along with concept level contingency. Additional details are contained in the Supplement.

Estimated Mitigation Construction Costs:

1 – Central Utility Plant (CUP)	**$16M**
3 – Concourse C	
3 - C1	**$14M**
3 - C2	**$31M**
3 - C3	**$36M**
4 – Terminal, building sections	
4 T1 – Ticket Lobby	**$47M**
4 T2 – South Node	**$36M**
4 T3 – Oregon Market Place – South	**$20M**
Total	**$200M**

Figure 4

General Next Steps

Seismic mitigation inside existing, operating facilities is inherently disruptive and expensive. For those reasons, a relatively high level of effort in the design phase of any seismic strengthening project not only ensures that the desired objectives are achieved in the most efficient manner; but can save significant construction cost and time, as well as reducing passenger inconveniences for work inside the Terminal. This should include consideration of the following:

- Detailed geotechnical site assessments to confirm design seismic response spectra, as well as local liquefaction effects on both free-field ground deformations and pile supported foundations.
- Consideration of higher-level structural analysis, such as non-linear, or non-linear time-history computer analyses in order to obtain the most accurate picture of the expected structural behavior.
- Assessment of MEP systems critical for function in the Immediate Occupancy scenario, including site surveys of existing support and bracing conditions. This may include adjacent non-critical systems that could impact the critical system during a seismic event.
- Architectural review of exterior enclosure systems and drift compatibility, along with interior systems critical for function or potential falling hazards, for the Immediate Occupancy scenario.
- Refinement of proposed projects for more accurate pricing and scheduling, including review in the context of broader PDX master planning.
- Utility and other lifelines on a regional scale.

Building Assessments and Mitigation

Following are assessments of the key building assets identified by the Port relative to their expected performance in a Code level seismic event considering their current condition. The critical BI assets have also been assessed for the expected return-period earthquakes that correspond to Immediate Occupancy, Life Safety, and Collapse Prevention in the buildings' current conditions.

For the latter assessment, as well as mitigation design, it is recommended that ASCE 41-13 be used in lieu of the Oregon Structural Specialty Code (OSSC). While the basic objectives of the OSSC will need to be met, design for the enhanced performance objectives that are being considered by the Port may be better served by utilizing other codes that were specifically developed to analyze the performance of existing buildings. As an example, rather than the constant factors in the OSSC separating the performance objectives previously discussed (OSSC force factor between LS and CP, as well as between Essential Facility and LS, is 1.5), ASCE 41 provides specific factors for different elements of the system, and for each performance objective, that are unique to the material and system type, resulting in more accurate predictions of the structural performance. In the sections following, we present two different code evaluations. First, the current condition of the lateral force capacity of the buildings relative to current OSSC design with USGS earthquake mapping, including a brief description of the condition of the building relative to current Code ductile detailing requirements (i.e. minimum Building Department requirements). Second, the expected return period earthquake for the three performance objectives based on ASCE 41 criteria, considering the site specific response spectra that GRI and New Albion have presented in this study. It must be noted, however, that these evaluations of performance objectives vs. return period DO NOT address liquefaction effects – they would be as if all liquefaction concerns were mitigated, but the building not upgraded. Liquefaction effects and their effects on performance are discussed in the sections for each building.

For the critical BI assets, a description, along with a schematic plan layout, of a possible mitigation scheme is discussed, including a rough order of magnitude cost estimate. The Supplement contains additional detail of the estimated cost. The intent of the mitigation strategy is to achieve as close as practical to an Immediate Occupancy performance.

For all of the buildings, a brief "Next Steps" describes further assessments or analyses that are recommended with respect to mitigating the seismic hazards.

Central Utility Plant (CUP)

Building ID#: 1

Building Address: 7110 NE Airport Way, Portland, OR 97218

Building Description: The CUP is a one-story structure housing the airport's central plant, including emergency utility systems, with a partial two-story office and storage section.

Building Structural System: The gravity system is steel framing with concrete over metal deck at the second floor area and metal deck at the roof. The ground floor is a structured slab-on-grade designed for high gravity loads. Foundations are timber piles (original construction) and auger-cast piles (subsequent constructions). Lateral system consists of a combination of steel concentric braced frames, wire rope concentric braced frames, and steel truss moment frames.

Code Summary: Originally constructed under the 1970 UBC, expanded in 1992, and seismically upgraded according to the 1994 UBC (1996 OSSC), with Hazardous Facility Importance Factor of 1.25.
Seismically upgraded shear design Code = 65% base shear of current Code in the E-W direction, and 87% of base shear of current Code in N-S direction for a Hazardous Facility. Uses "ordinary" braced frames and moment frames are no longer permitted by Code in this seismic zone.

General Seismic Performance: The CUP design under the 1994 Code lacks the current Code requirements for ductile systems and detailing. The Code has increased the required design force for a steel ordinary braced frame substantially, resulting in the design force being between 65% and 87% of current Code requirements. The lack of ductile detailing in the lateral elements is expected to result in greater localized damage. Even with a relatively flexible roof diaphragm, the disparity in systems (cable tension bracing, HSS concentric bracing, and steel truss moment frames) is likely to result in undesirable distributions of lateral force. The cable braces and moment frames may see deflections exceeding those desirable for rigid building attachments to stiff exterior masonry work, and for glazing systems, unless properly detailed for the larger deflections, especially to achieve Immediate Occupancy. The exterior masonry walls appear especially susceptible due to the thin sections and relative lack of ductility.

The evaluation of the existing structure using ASCE 41 performance criteria and site specific response spectra developed for this study, results in the following estimates of return period earthquake for the three performance levels considered (without consideration of liquefaction effects) as shown in Figure 5:

Immediate Occupancy:	110 years
Life Safety:	270 years
Collapse Prevention:	680 years

MEP equipment and systems are seismically braced and would be expected to behave satisfactorily for an essential-facility level design, except for the effects of excessive building deflection on connections. There are parts storage racks that do not appear to be braced; but do not appear that they would affect the immediate operations.

The site is susceptible to liquefaction during a seismic event. The building piles, according the the existing drawings, are relatively consistent in that they only penetrate the denser sand layer approximately five feet. It is anticipated that this will allow settlement of several inches resulting from liquefaction. This will cause distress, particularly in the exterior wall systems and MEP connections. The structured slab-on-grade should mitigate the effects of the larger free-field soil settlement to some extent, though unlikely to Immediate Occupancy levels.

Mitigation: Mitigation will necessarily address both the liquefaction issues along with the seismic strength and ductility of the lateral force resisting system (LFRS). Potential mitigation schemes are identified below. Replacement of the facility is not addressed as an option, but should be considered by the Port. Refer to Figure 6 for proposed concept mitigation scheme.

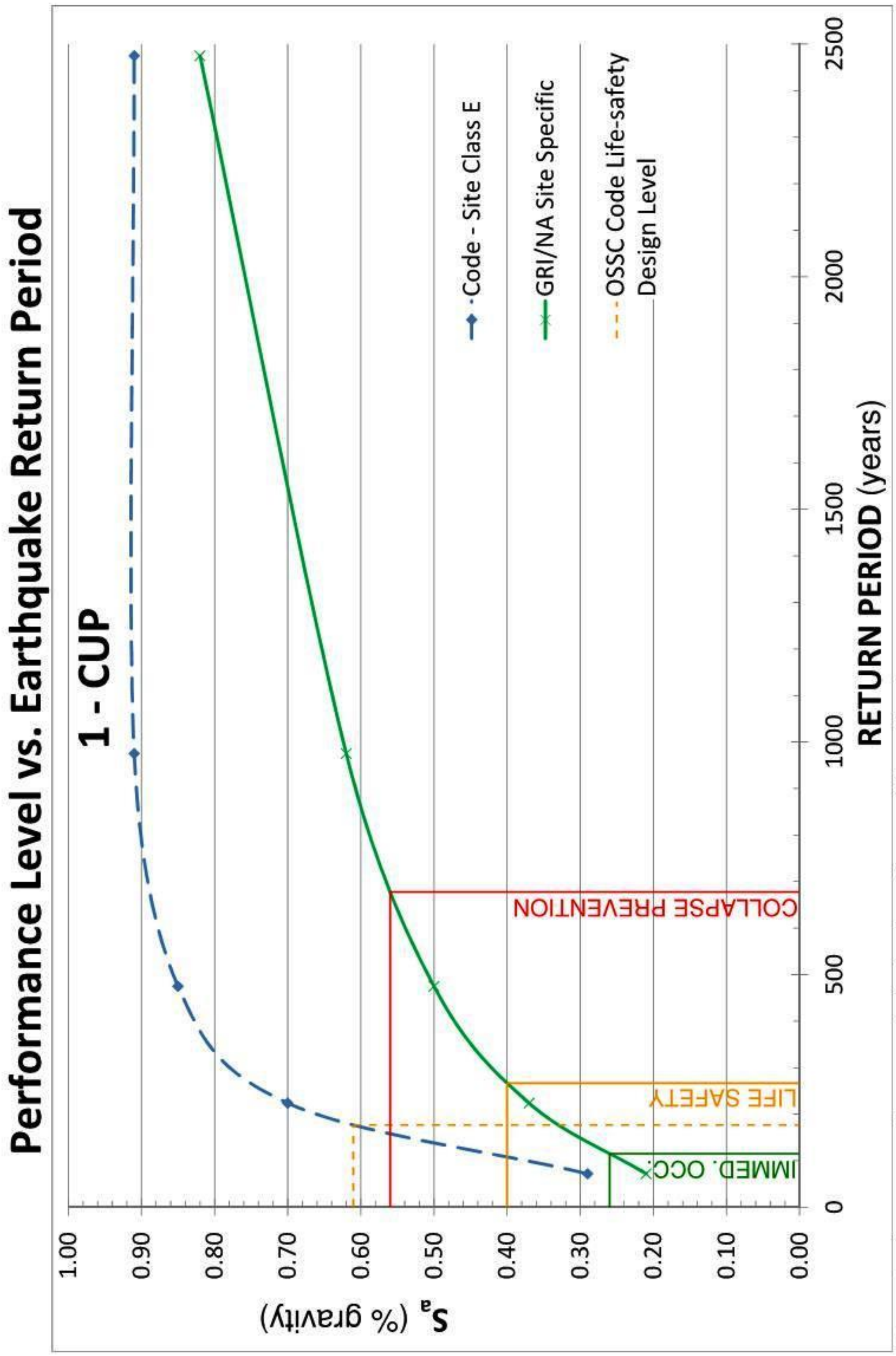
Liquefaction: Mitigation of the apparent deficiencies in resisting the effects of liquefaction settlement should be addressed, by first commencing more in-depth geotechnical and structural studies to more precisely understand the magnitude of expected settlement and resulting effects on the foundations, slab-on-grade, and other building systems. This could be mitigated by adding micropiles at all of the building columns and at critical locations of equipment, including the equipment yard. A survey of utilities entering the building should be made, and flexible connections installed at all locations of dissimilar support conditions, e.g. below grade that may settle while entering a pile-supported building.

Lateral Force Resisting System: To reduce the risk of drift-related issues – MEP connections, masonry brittleness – a concrete shear wall scheme would be an appropriate solution. This could consist of replacing existing exterior wall bays with new concrete shear wall and/or adding external flying buttresses. A more detailed analysis of the roof and mezzanine diaphragms would determine the trade-off of wall spacing vs. diaphragm reinforcing and drag ties. Due to the non-ductile nature of the typical existing exterior wall comprising of 4" concrete masonry units with brick veneer, consideration should be given to replacing that with a more drift-tolerant system, such as metal studs with metal panel. Essential MEP systems should be reviewed for support, and attachments relying on the thin masonry elements should be replaced.

Utility Tunnel: There is approximately 80 feet of buried, corrugated-steel-pipe (CSP) tunnel between the pile-supported CUP and the pile-supported tunnels under the HQ/P2 buildings. The issues of settlement of this portion of tunnel, including the connections to non-settling structures at each end, could be mitigated by replacing this tunnel with a new, pile-supported tunnel. It may be feasible to excavate and encase the existing tunnel within the new tunnel, then reconnecting utility supports to the new tunnel as demolition of the existing CSP progresses, causing minimal disruption to services. It should be noted that there are other existing, non-pile-supported utility tunnels on the Terminal side of HQ/P2. These are beyond the scope of this report, but may require similar addition of pile support, and should be considered in a comprehensive utility mitigation scheme.

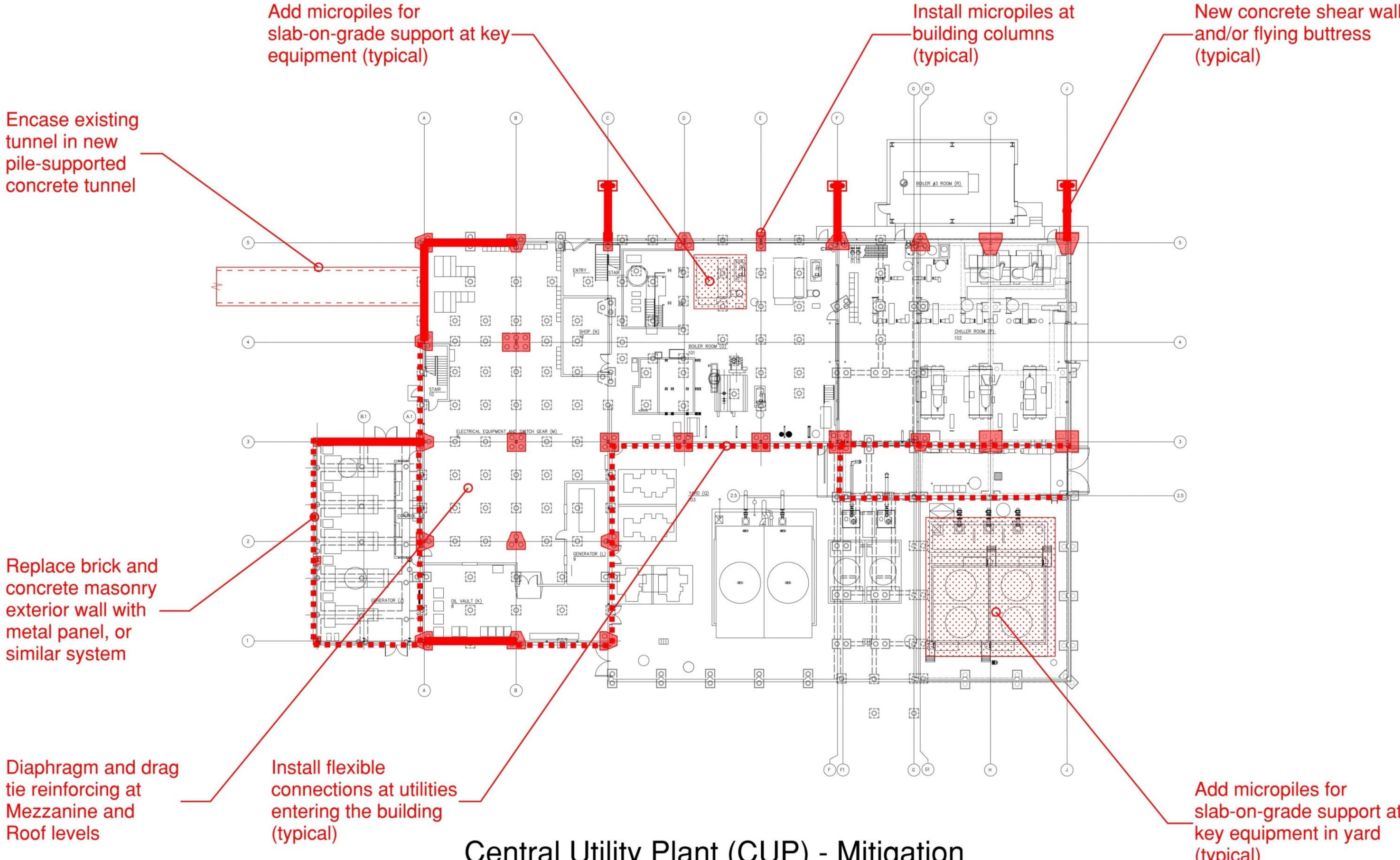
Construction Cost Estimate: $16,000,000.

Next Steps: Detailed geotechnical, structural, and MEP assessments to verify the liquefaction effects, site response spectra, and assess MEP systems that are critical for Immediate Occupancy. Refined assessments to determine the preferred mitigation scheme.



Note: Performance does not include the effects of liquefaction.

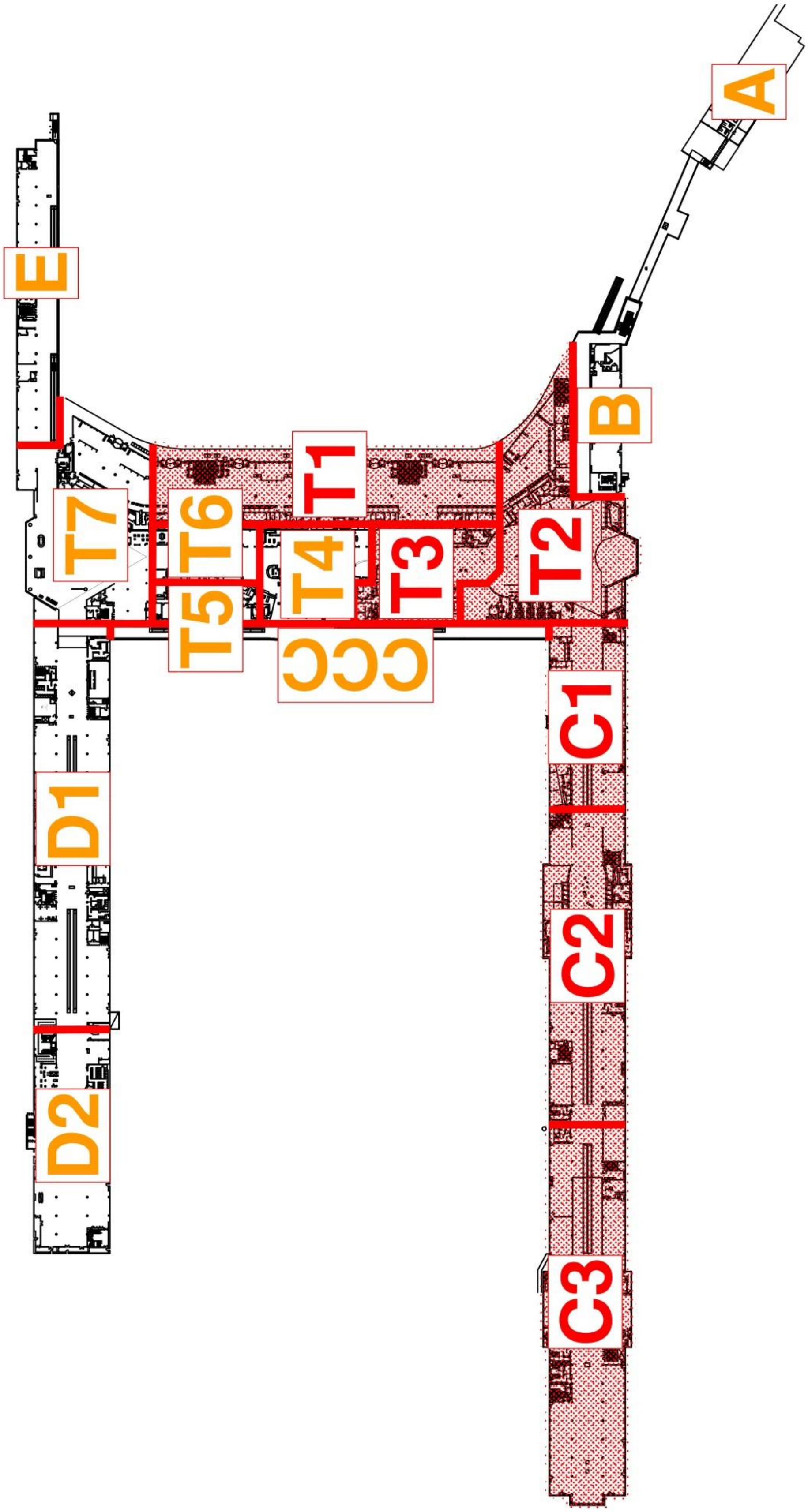
Figure 5



Central Utility Plant (CUP) - Mitigation

Figure 6

Terminal Building Layout

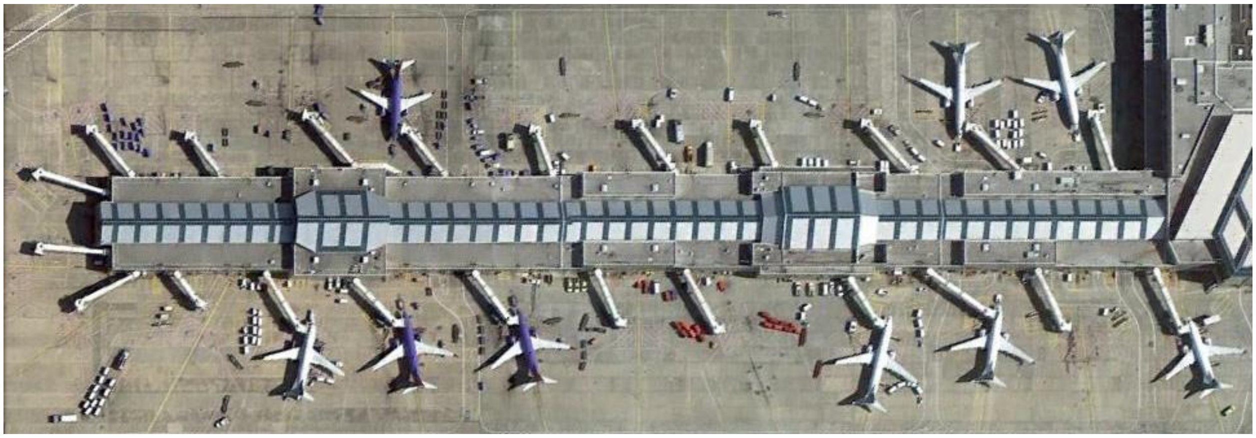


kpff

Concourse C

Building ID#: 3 – C1, C2, C3

Building Address: 7000 NE Airport Way, Portland, OR 97218



Building Description: Concourse C is a two-story structure with roof mechanical penthouses. It is divided into three separate structures, each with similar construction, but separated by seismic joints. Concourse C was constructed as part of the Terminal Expansion South projects in the late 1990's.

Building Structural System: Gravity system is steel framing with composite concrete decks on all levels. Ground floor is slab-on-grade with integral grade beams. Foundations are steel piles. Lateral system is steel special moment-resisting frames.

Code Summary: Constructed under the 1994 UBC (1996 OSSC).
Base shear design Code = 103% base shear of current Code.
Steel special moment-resisting frames.

General Seismic Evaluation: Concourse C was constructed in the latter 1990's, using the relatively new "dog-bone" moment frame detailing derived from research following the 1994 Northridge earthquake. Detailing is generally consistent with current Code ductility requirements and the design seismic force meets current Code for Seismic Occupancy Category III, which is enhanced life-safety criteria, between standard occupancy and Essential Facility. Lateral drifts of the building for Immediate Occupancy in a 475-year earthquake will exceed current detailing for drift of exterior components and seismic joints, along with interior architectural and utility components. Exterior metal panels are a flexible system; however damage to glazing should be anticipated.

The site is susceptible to liquefaction, which may cause ground settlements exceeding one foot for return period events over 1000 years, and approximately eight inches at the 475-year event. This will cause significant distress in the slab-on-grade and utility tunnel that rely on soil support, along with architectural and MEP elements that rely on the slab and tunnel for their support. The pile foundations are only expected to settle approximately one inch at a long return period event.

The evaluation of the existing structure using ASCE 41 performance criteria and site specific response spectra developed for this study, results in the following estimates of return period earthquake for the three performance levels considered (without consideration of liquefaction effects) as shown in Figure 7:

Immediate Occupancy:	130 years
Life Safety:	1000 years
Collapse Prevention:	2200 years

It should be noted that while the Concourse C moment frames are expected to perform well for OSSC Occupancy Category III, ASCE 41 criteria for moment-resisting frames at the Immediate Occupancy performance level are much more stringent than the OSSC. This is due in large part to steel moment frames being a relatively flexible system, and ASCE 7 differentiates between the lateral systems and materials used more specifically than the OSSC. These deflections primarily affect the integrity of the secondary structural systems – architectural elements and MEP systems. While these systems may be allowed to undergo substantial damage at a Life-safety level, they must remain relatively intact, or even operable, to achieve an Immediate Occupancy performance.

MEP equipment and systems are generally braced and expected to behave satisfactorily for an enhanced life-safety level design, except where affected by liquefaction settlement.

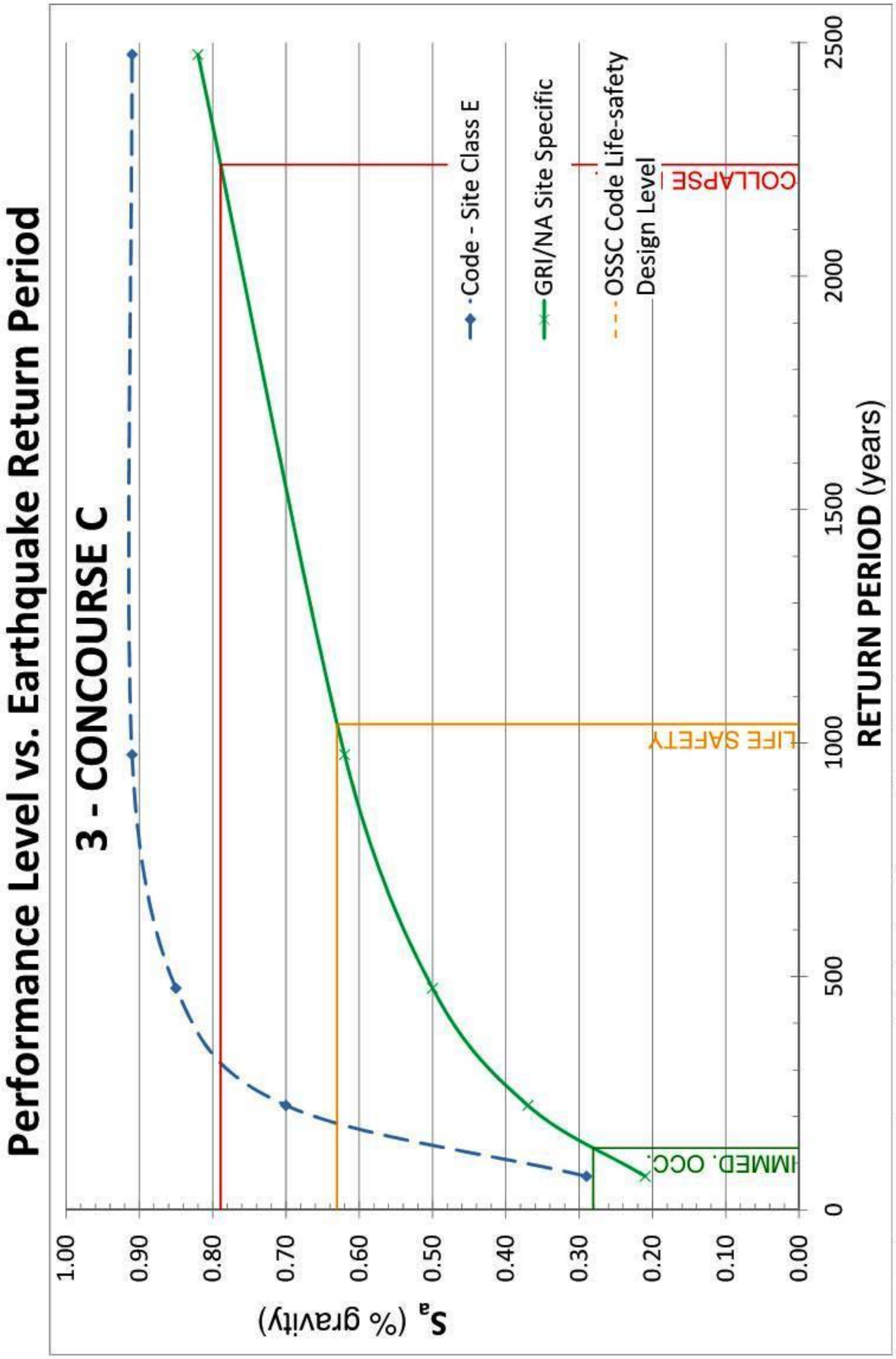
Mitigation: Mitigation will necessarily address both the liquefaction issues of the slab-on-grade, along with the seismic strength and ductility of the lateral force resisting system (LFRS). Potential mitigation schemes are identified below. Refer to Figure 8a and 8b for proposed concept mitigation scheme.

Liquefaction: The piles for Concourse C are deep enough that settlement of the building is not expected to exceed approximately one inch, even at long return period earthquakes. However, the slab-on-grade and utility tunnel rely on soil support, which is expected to experience settlements that could exceed one foot. To mitigate this, one of, or a combination of, the following could be used. Install micropiles under the existing slab-on-grade to reduce the effective span to approximately 10 feet. Note that prior to applying heavy point loads (e.g. bag cart tugs) after a seismic event resulting in liquefaction, filling voids under the slab with a low-density, pumpable grout will be necessary. Alternatively, the number of micropiles may be reduced (not likely eliminated) with the addition of a bonded, reinforced concrete topping, approximately 4-inch thick, over the existing slab-on-grade, to create a thicker, two-way slab. This may be more practical in the open areas, such as the east end baggage make-up. The utility tunnel should be mitigated by adding micropiles alongside the tunnel to minimize impacts to the existing utilities.

Lateral Force Resisting System: An increase in both strength and stiffness of the existing moment-frame system is necessary to achieve a performance level of Immediate Occupancy. Reinforcing the existing moment frames to achieve this would be very intrusive and disruptive. In lieu of reinforcement, a damping system that reduces the demand on the existing system may be effective, more economical, and less disruptive. A system of fluid viscous dampers could be installed in diagonally braced bays, and would absorb a significant amount of the seismic demand, and help keep the displacements within acceptable limits of the current construction. Notably, these have significant latitude in placement, including not requiring alignment from floor to floor, which could be important with the desire to keep the spaces as open as possible. Also, the nature of these systems is that the forces are greatest in them at peak velocity of the building (zero displacement) which is out of phase with the demand on the existing moment frames, whose force is greatest at maximum displacement (zero velocity). This will greatly reduce, or eliminate, the need to reinforce the columns and foundations. Further analysis will be required to determine quantity of braces required and optimal layout. Fluid viscous dampers have been used successfully both locally and along the west coast for both new and retrofit applications.

Construction Cost Estimate: $81,000,000.

Next Steps: Detailed geotechnical, structural, and MEP assessments to verify the liquefaction effects, site response spectra, and assess MEP systems that are critical for Immediate Occupancy. Refined assessments to determine the preferred mitigation scheme for liquefaction. Full structural analysis of the LFRS, which may include non-linear or time-history non-linear analyses to most accurately evaluate the structure and provide upgrades that achieve the desired performance while optimizing economy.



Note: Performance does not include the effects of liquefaction.

Figure 7

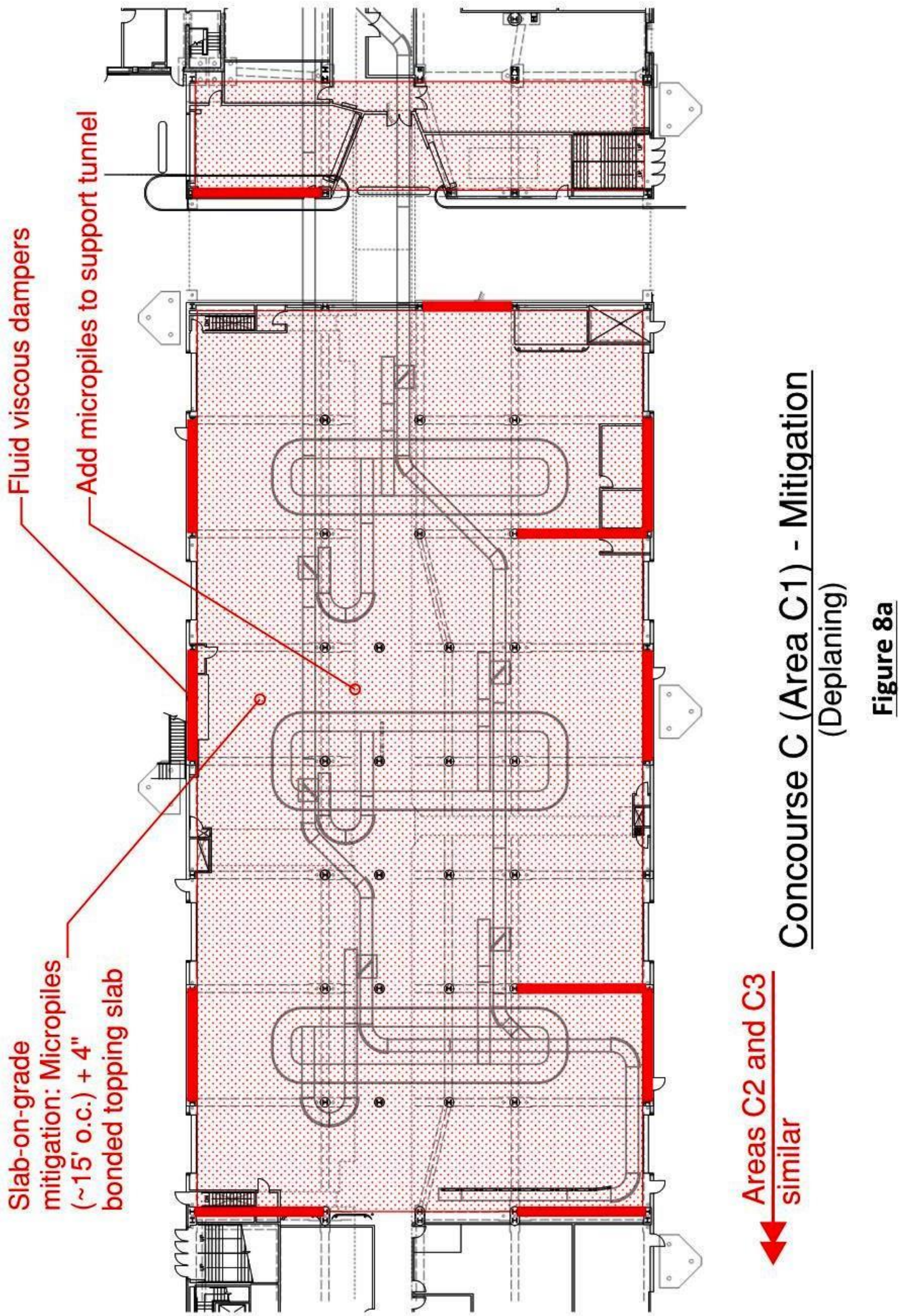

Fluid viscous dampers
Add micropiles to support tunnel
Slab-on-grade mitigation: Micropiles (~15' o.c.) + 4" bonded topping slab
Areas C2 and C3 similar
Concourse C (Area C1) - Mitigation
(Deplaning)
Figure 8a

Figure 8a

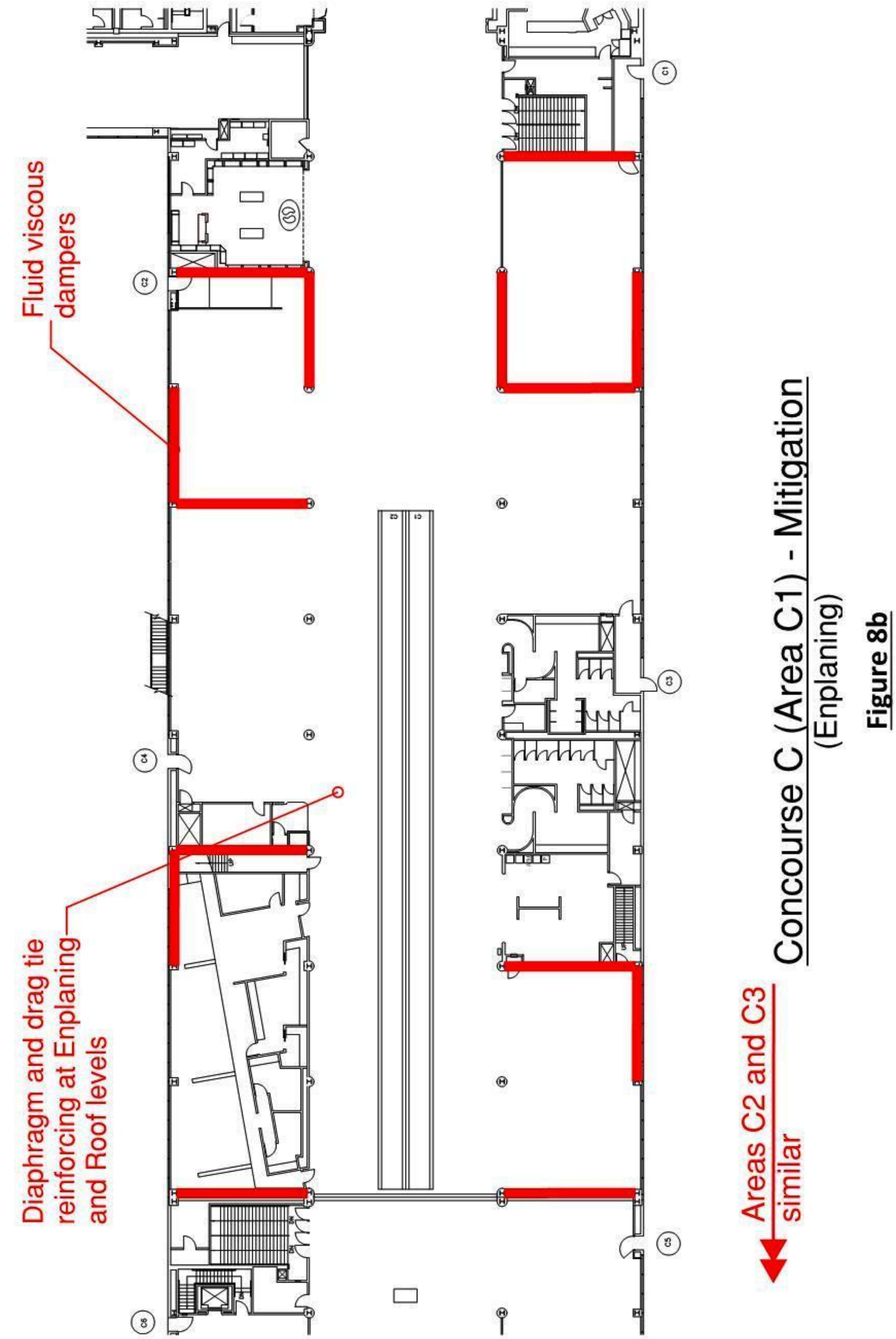

Fluid viscous dampers
Diaphragm and drag tie reinforcing at Enplaning and Roof levels
Areas C2 and C3 similar
Concourse C (Area C1) - Mitigation
(Enplaning)
Figure 8b

Figure 8b

Terminal – Ticket Lobby

Building ID#: 4 - T1

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The Ticket Lobby is a two-story structure with a narrow mechanical mezzanine along the west edge.

Building Structural System: Gravity framing is steel framing with concrete on metal deck (mostly non-composite) at the Enplaning level, and open-web steel joists with metal deck at the roof. Ground floor is slab-on-grade. Foundations are steel piles. The original construction is from 1973 and this portion of the terminal was extensively modified in the mid-90's by the Terminal Access Program (TAP); including the addition of two concrete elevator/escalator/stair cores and two pedestrian tunnels to the parking garage. The seismic upgrade included these cores along with additional shear walls and steel braced frames. The steel braced frames occur above the mezzanine level only. In the east-west direction, steel braced frames are used only to brace the mezzanine up to the roof level – all primary lateral elements are concrete shear walls.

Code Summary: Constructed/upgraded under the 1991 UBC (1993 OSSC).

Base shear design Code = 97% base shear of current Code for concrete shear wall construction.
Base shear design Code = 63% base shear of current Code for steel concentric braced frame construction.
Shear wall reinforcing and steel braced frame detailing is considered "ordinary" criteria, which is no longer a permitted construction type for new buildings in this seismic region; although shear wall detailing appears to be close to current Code "special" criteria.

General Seismic Evaluation: The Terminal Ticket Lobby seismic upgrade consists primarily of concrete shear walls; however, steel braced frames were used from the existing mezzanine up to the roof level. The required seismic design force for ordinary steel frames has increased significantly since these were constructed, resulting in a current Code force significantly larger than the design. A lack of current Code ductile detailing in the braced frames and shear walls may result in greater localized damage; though these effects are expected to be less pronounced in a shorter, stiffer building. Site is susceptible to liquefaction during a seismic event. The existing drawing indicate that the original piles do not penetrate the denser sand layer adequately, and it is estimated that the piles may settle several inches in a seismic event. This will cause distress in the building framing and floor systems, along with damage to the secondary structural systems. The seismic upgrade work performed in the Terminal Access Program (TAP) project utilized micropiles to withstand the seismic forces at the new shear walls. These micropiles are deep enough that they are expected to experience very small settlement, which may lead them to become overloaded at the adjacent older piles settle and shed the building loads to them. The slab-on-grade may see liquefaction settlement of one foot or more, which will result in damage to architectural and MEP systems supported by the slab as well as the exit vestibules.

The evaluation of the existing structure using ASCE 41 performance criteria and site specific response spectra developed for this study, results in the following estimates of return period earthquake for the three performance levels considered (without consideration of liquefaction effects) as shown in Figure 9:

Immediate Occupancy:	600 years
Life Safety:	920 years
Collapse Prevention:	1900 years

MEP equipment and systems are generally braced and expected to behave satisfactorily for an enhanced life-safety level design, except where affected by liquefaction settlement; however, the systems vary greatly in age and several may remain that were installed prior to newer bracing requirements.

Mitigation: Mitigation will necessarily address both the liquefaction issues along with the seismic strength and ductility of the lateral force resisting system (LFRS). Potential mitigation schemes are identified below. Refer to Figure 10a, 10b and 10c for proposed concept mitigation scheme.

Liquefaction: The settlement of the building columns and structural walls due to settlement of the existing older piles may be addressed by the addition of new micropiles at each column. This would involve slab demolition as well as penetrating the existing pile caps to install the new piles without major disruption of airport functions. These pile caps are identified in the plan following. The slab-on-grade may be mitigated by removing and replacing with a structural slab that spans to the reinforced pile caps, or by relatively closely spaced micropiles supporting the existing slab-on-grade. A combination of these methods may be appropriate, depending on existing constraints. Sequencing the slab mitigation may also be possible, for example, mitigating areas supporting critical MEP equipment over open assembly space. It should be noted that the entry vestibules currently rely on soil bearing support, rather than piles, and thus should be prioritized for mitigation.

Lateral Force Resisting System: The LFRS is primarily concrete shear walls, which are expected to perform adequately for Immediate Occupancy for the 475-year seismic event. Above the Mezzanine along the west edge of this area are concentric steel braced frames, bracing both the Mezzanine to the Roof, and the Roof down to the shear walls below. These braces can be replaced with more ductile buckling-restrained braces, which have advantages in ductility performance, along with limiting the force that will be transferred to the columns. It should be anticipated that certain drag and diaphragm connections will also require reinforcing. Additional piles to resist seismic overturning forces may be required, as the acceptance criteria for these elements is significantly higher for Immediate Occupancy. This will require consideration, but may in part be in conjunction with adding piles to eliminate gravity column settlements.

Construction Cost Estimate: $47,000,000.

Next Steps: Detailed geotechnical, structural, and MEP assessments to verify the liquefaction effects, site response spectra, and assess any MEP systems that may be critical for Immediate Occupancy. Structural analyses may include more intensive non-linear or time-history non-linear analyses to most accurately evaluate the structure and provide upgrades that achieve the desired performance while optimizing economy. The Roadway Canopy should be reviewed for the anticipated seismic loads and lateral displacements of the Parking Garage and Terminal Buildings, as well as internal deformations as they may affect the glazing. As the P1 Parking Garage supports the east side of the Roadway Canopy, evaluation of that structure for a compatible performance object should be included.

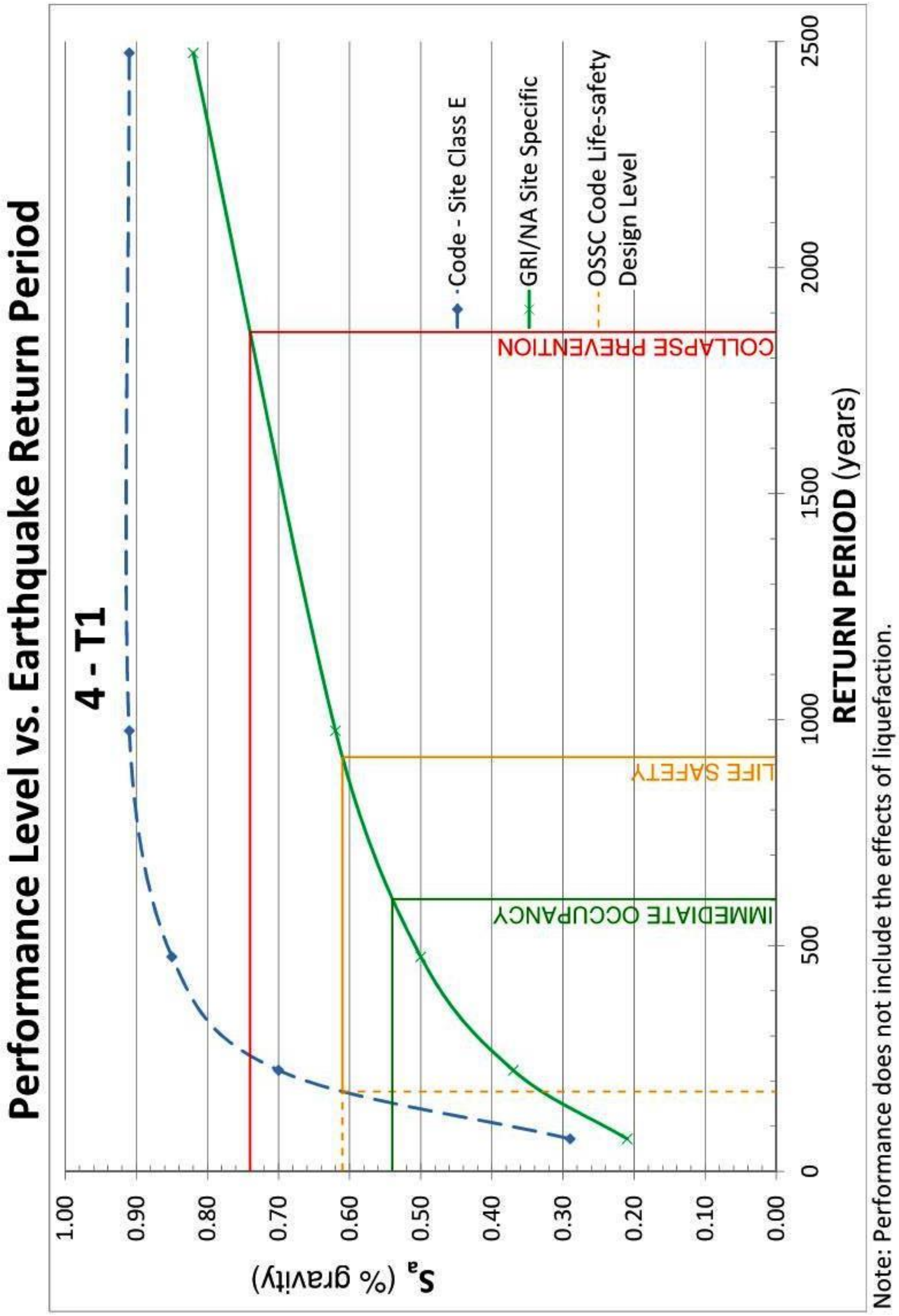

Performance Level vs. Earthquake Return Period
4 - T1
Code - Site Class E
GRI/NA Site Specific
OSSC Code Life-safety Design Level
COLLAPSE PREVENTION
LIFE SAFETY
IMMEDIATE OCCUPANCY
S_a (% gravity)
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
500
1000
1500
2000
2500
RETURN PERIOD (years)
Note: Performance does not include the effects of liquefaction.

Figure 9

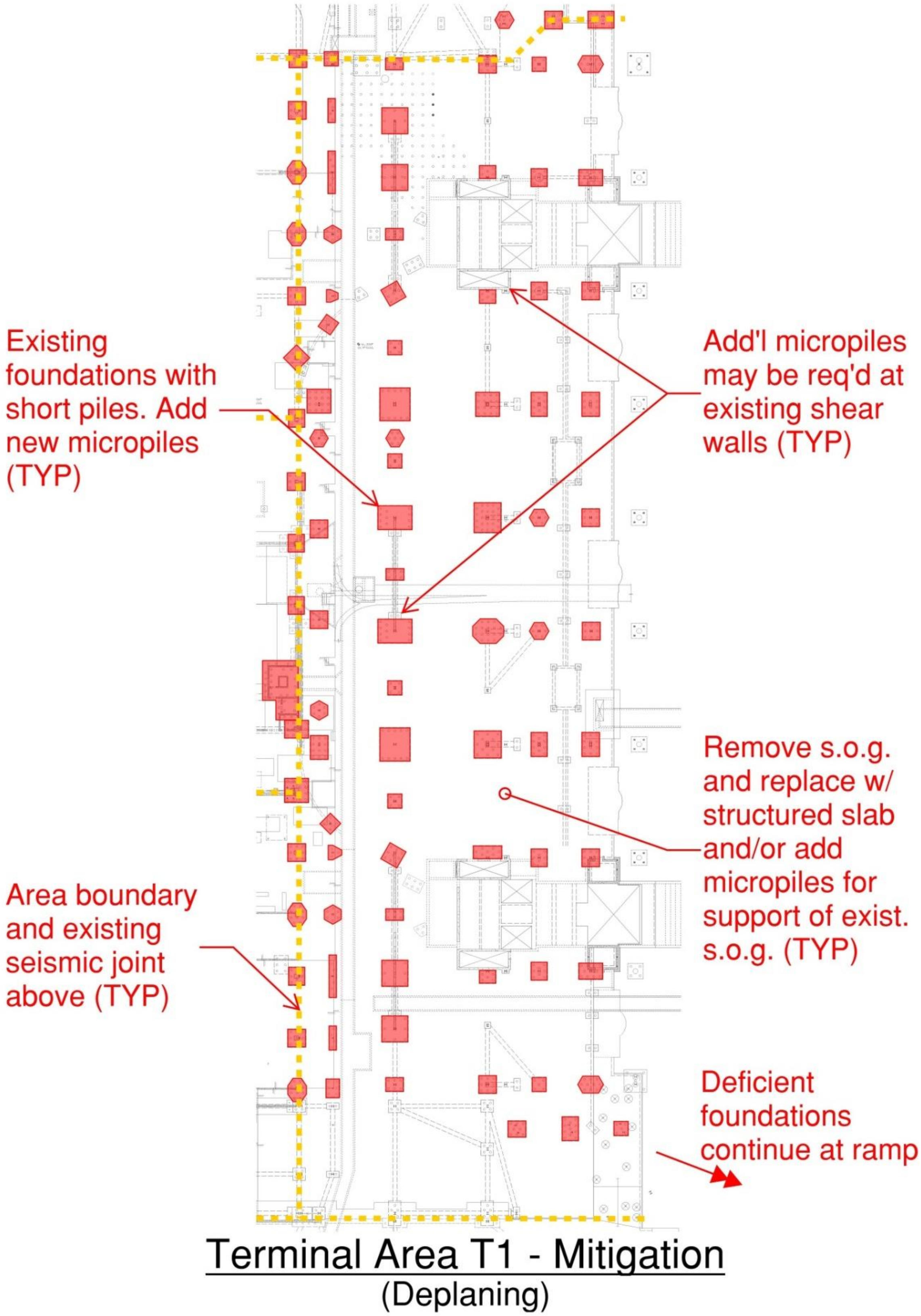
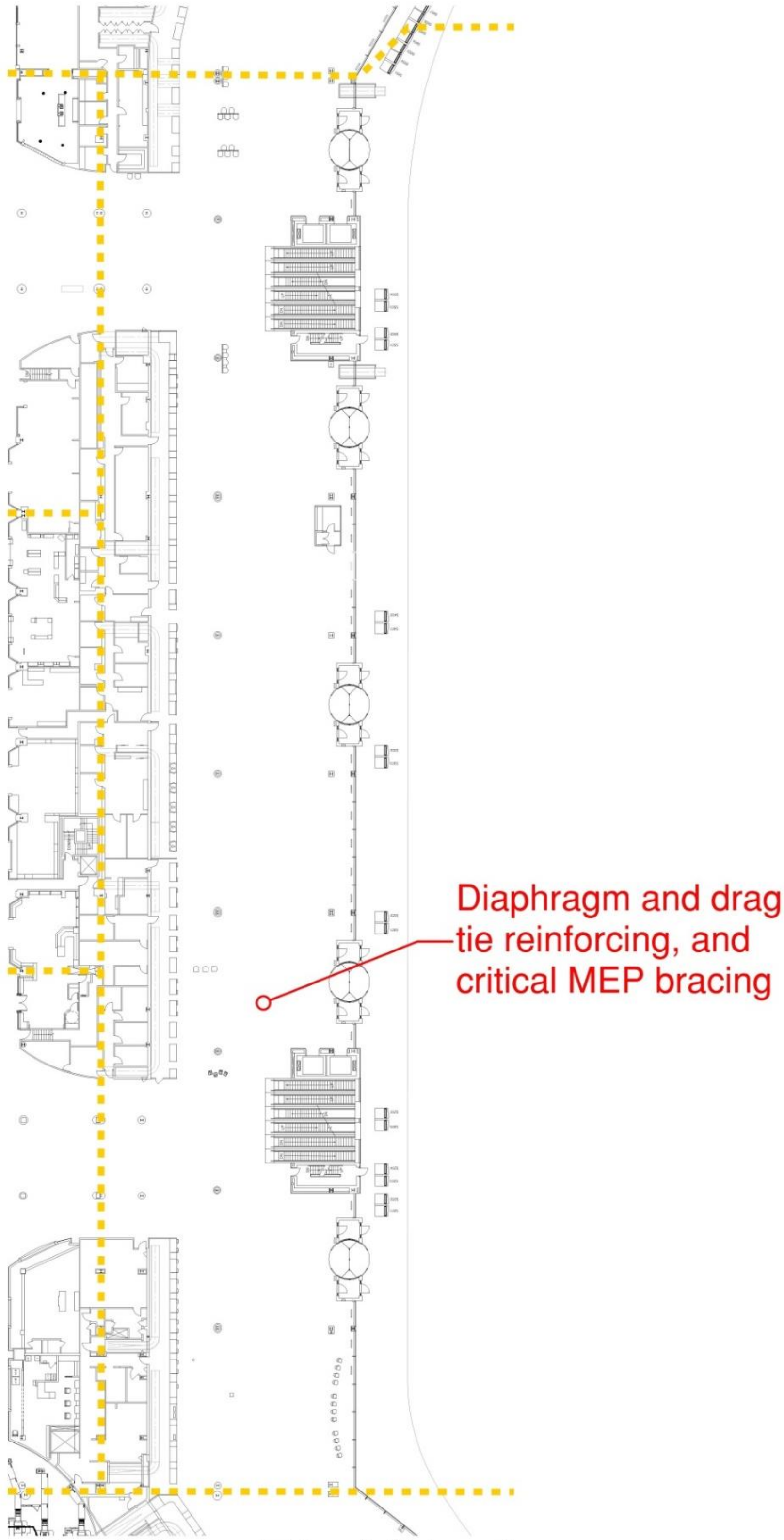
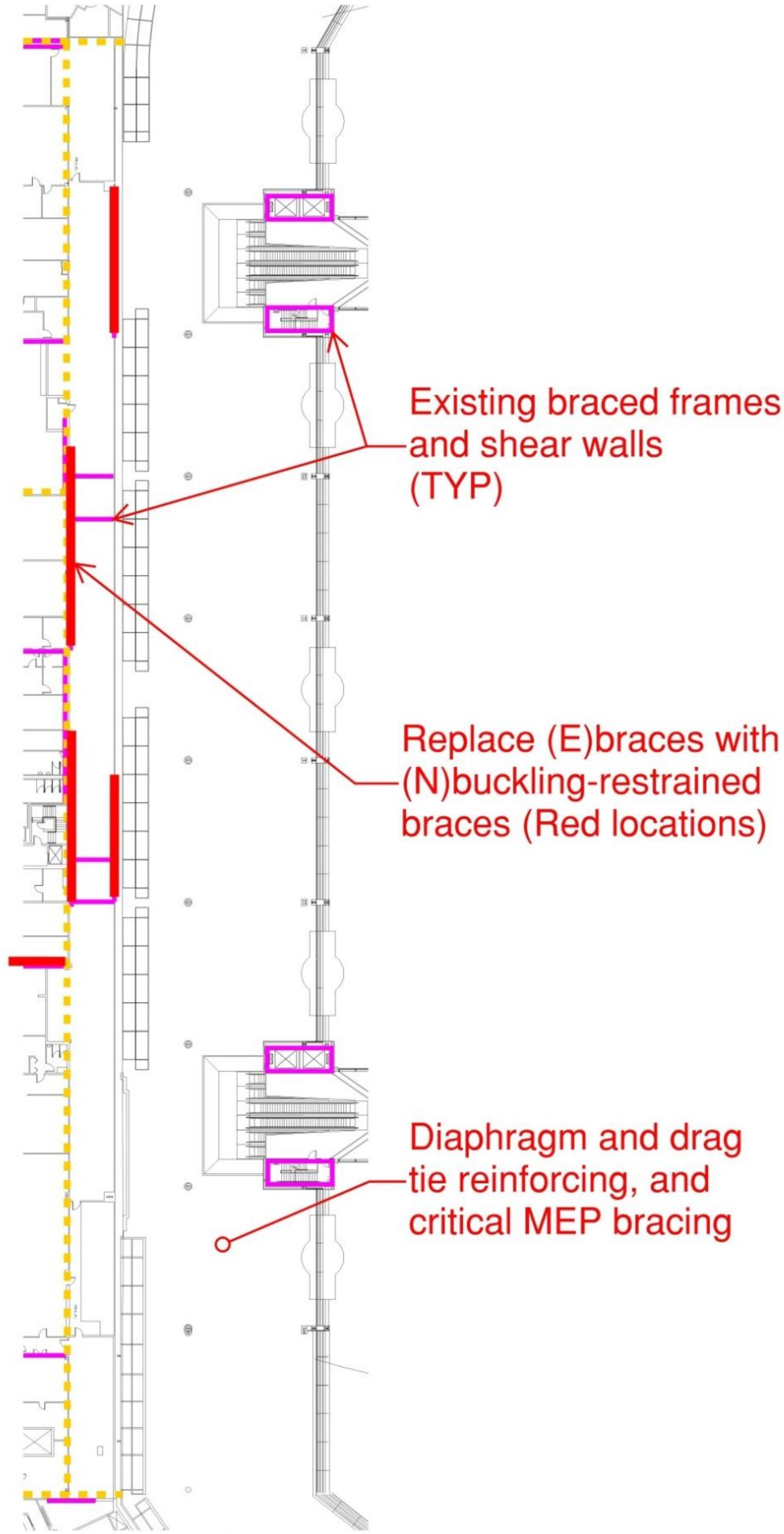

Existing foundations with short piles. Add new micropiles (TYP)
Add'l micropiles may be req'd at existing shear walls (TYP)
Remove s.o.g. and replace w/ structured slab and/or add micropiles for support of exist. s.o.g. (TYP)
Area boundary and existing seismic joint above (TYP)
Deficient foundations continue at ramp
Terminal Area T1 - Mitigation
(Deplaning)

Figure 10a



Terminal Area T1 - Mitigation
(Enplaning)

Figure 10b



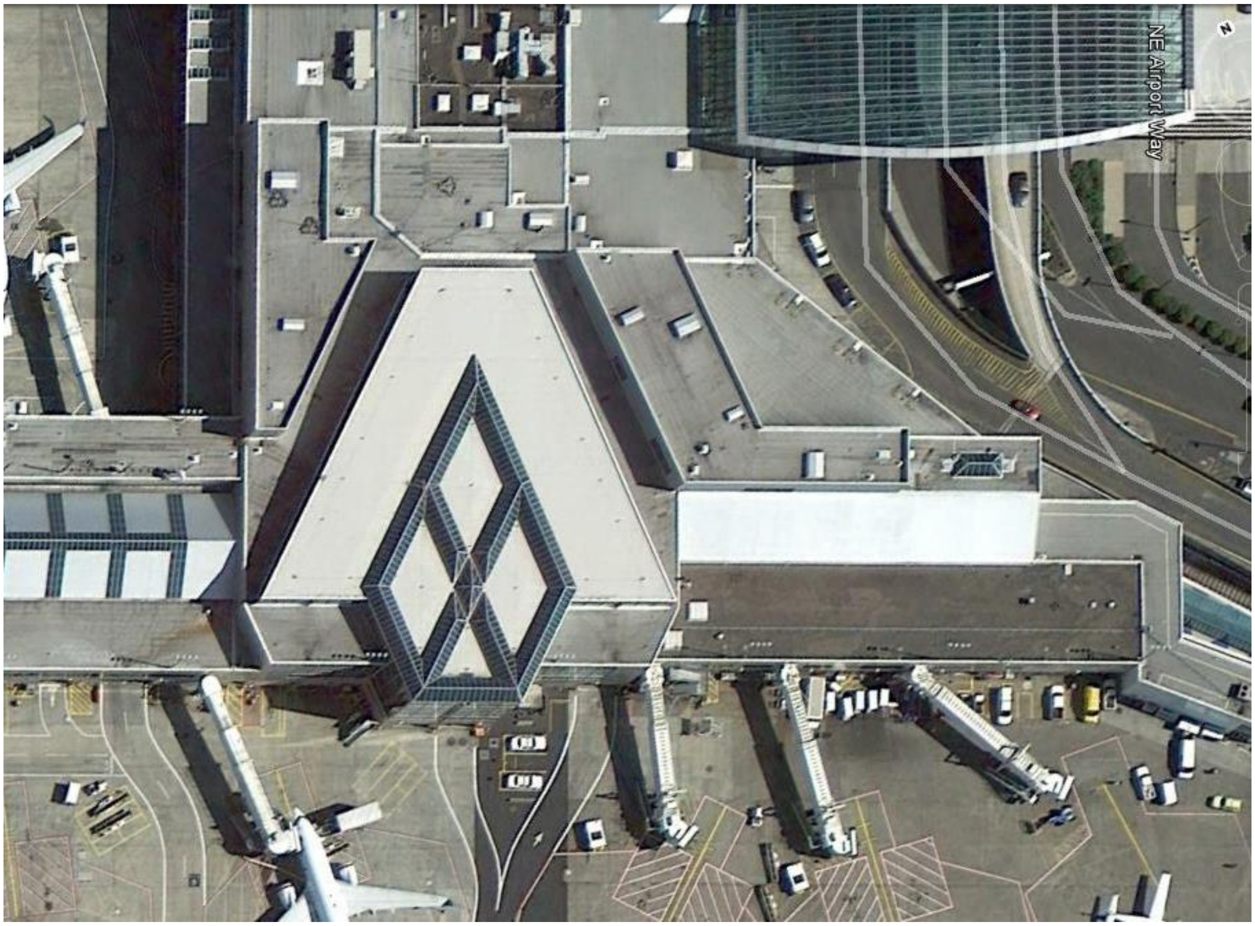
Terminal Area T1 - Mitigation
(Mezzanine)

Figure 10c

Terminal – South Node

Building ID#: 4 - T2

Building Address: 7000 NE Airport Way, Portland, OR 97218



Building Description: The south node of the terminal is a three-story structure with mechanical penthouses on the roof. The central area is a two-story space with a steel space-frame roof structure.

Building Structural System: Gravity system is steel framing with composite concrete decks on all levels. Ground floor is slab-on-grade. Foundations are steel piles. Lateral system is reinforced concrete bearing/shear walls.

Code Summary: Constructed under the 1994 UBC (1996 OSSC).
Base shear design Code = 103% base shear of current Code.
Shear wall reinforcing detailing is considered "ordinary" criteria, which is no longer a permitted construction type for new buildings in this seismic zone; however detailed analysis may demonstrate compliance with most "special" criteria.

General Seismic Performance: Terminal South Node meets the current Code for design force level for an Occupancy Category III building. However, it likely lacks some of the newer concrete reinforcing ductility detailing requirements of current Code. A lack of ductile detailing in shear wall buildings may result in greater localized damage; though these effects are expected to be less pronounced in a shorter/stiffer building. The site is susceptible to liquefaction, which may cause ground settlements exceeding one foot for return period events over 1000 years, and approximately ten inches at the 475-year event, which will impact all systems relying on the slab for support. The pile foundations are expected to settle approximately one inch at a long return period event.

The evaluation of the existing structure using ASCE 41 performance criteria and site specific response spectra developed for this study, results in the following estimates of return period earthquake for the three performance levels considered (without consideration of liquefaction effects) as shown in Figure 11:

Immediate Occupancy:	1700 years
Life Safety:	2500 years
Collapse Prevention:	>2500 years

MEP equipment and systems are generally braced and expected to behave satisfactorily for an enhanced life-safety level design, except where affected by liquefaction settlement.

The Concourse B roof and adjacent walkway are seismically separated from the remainder of the structure and were not explicitly reviewed.

Mitigation: Mitigation will necessarily address both the liquefaction issues of the slab-on-grade along with ductility of the lateral force resisting system (LFRS). Potential mitigation schemes are identified below. Refer to Figure 12 for proposed concept mitigation scheme.

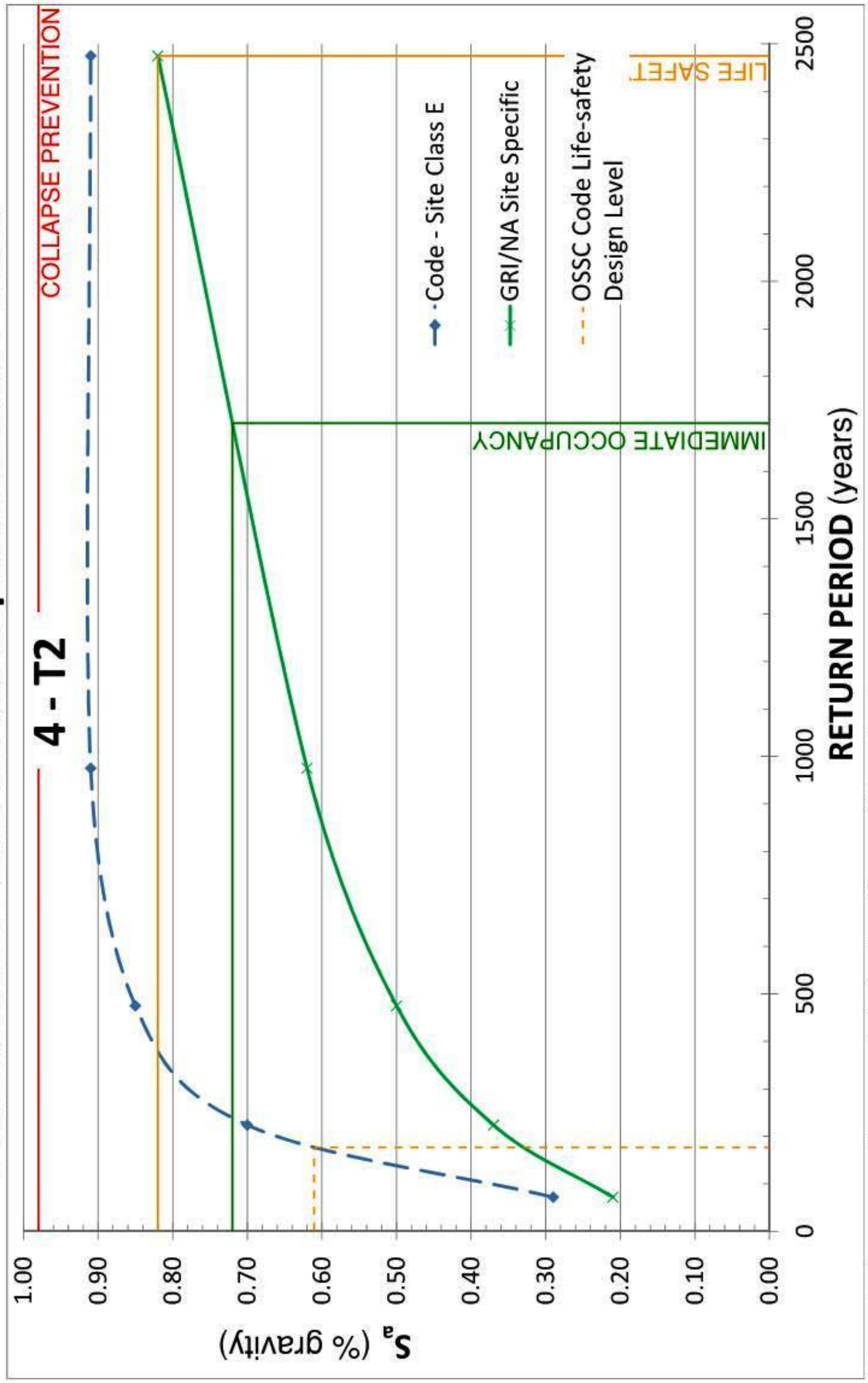
Liquefaction: The piles for the South Node are deep enough that settlement is not expected to exceed approximately one inch, even at long return period earthquakes. However, the slab-on-grade and utility tunnel rely on soil support, which is expected to experience settlements that could exceed one foot. To mitigate this, one of, or a combination of, the following could be used. Install micropiles under the existing slab-on-grade to reduce the effective span to approximately 10 feet. Note that prior to applying heavy point loads (e.g. bag cart tugs) after a seismic event resulting in liquefaction, filling voids under the slab with a low-density, pumpable grout will be necessary. Alternatively, the number of micropiles may be reduced (not likely eliminated) with the addition of a bonded, reinforced concrete topping, approximately 4-inch thick, over the existing slab-on-grade, to create a thicker, two-way slab. This might be considered in the open areas, such as baggage make-up. Demolition of the existing slab-on-grade, and replacement with a thicker, two-way slab, could also be considered where achievable considering the increased disruption. Either of these schemes would be combined with adding micropiles to support the utility tunnel. These could be installed alongside the tunnel to minimize impacts to the existing utilities.

Lateral Force Resisting System: The shear wall system in the South Node has the capacity to meet Immediate Occupancy criteria; however, a detailed review will likely identify some drag connections and/or diaphragm connections that are lacking in either strength and/or ductility and should be reinforced. Additionally, it should be anticipated that additional piles to resist seismic overturning forces may be required, as the acceptance criteria for these elements is significantly higher for Immediate Occupancy.

Construction Cost Estimate: $36,000,000.

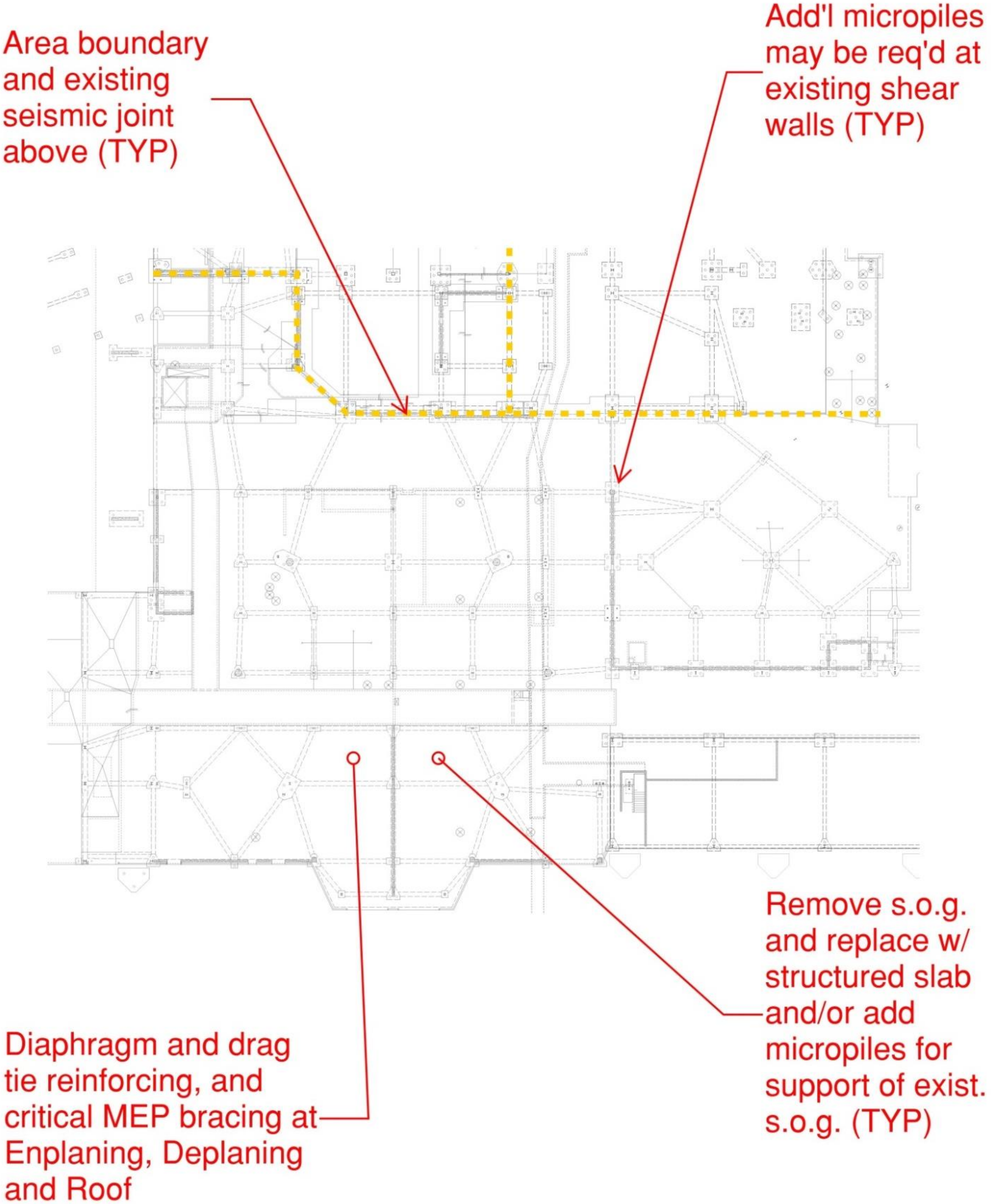
Next Steps: A detailed geotechnical assessment to verify the liquefaction effects and site response spectra. For Immediate Occupancy performance, a MEP assessment of the resilience of critical systems should be considered. If seismic strengthening of the South Node is considered, further structural analyses might include more intensive non-linear or time-history non-linear analyses to most accurately evaluate the structure and provide upgrades that achieve the desired performance while optimizing construction and interruption costs.

Performance Level vs. Earthquake Return Period



Note: Performance does not include the effects of liquefaction.

Figure 11



Terminal Area T2 - Mitigation
(Deplaning)

Figure 12

Terminal – Oregon Market Place South

Building ID#: 4 - T3

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The south portion of the Oregon Market Place is a three-story structure that combines several different construction projects. The mezzanine includes occupied spaces as well as primary mechanical space.

Building Structural System: The structure includes one-way concrete slab and beam construction from 1956, along with steel and composite deck construction from 1986 and 2002. The lateral system was completed with the Terminal Expansion South, Phase 2 and 3 (TES2 and TES3), projects. The lateral system is a combination of reinforced concrete shear walls and steel braced frames.

Code Summary: Started under the 1994 UBC (1996 OSSC), completed under the 1997 UBC (1998 OSSC).
Base shear design Code = 107% base shear of current Code for concrete shear wall construction.

Base shear design Code = 70% base shear of current Code for steel concentric braced frame construction.
Shear wall reinforcing and steel braced frame detailing are considered "ordinary" criteria, which is no longer a permitted construction type for new buildings in this seismic zone; however detailed analysis may demonstrate compliance with most "special" criteria for shear walls.

General Seismic Evaluation: Terminal-Oregon Market Place South is a combination of structures built over a span of almost 50 years. The structures have been tied together and seismically upgraded to perform as a single structure. The lateral system is a combination of concrete shear walls and concentric braced frames. The required seismic design force for ordinary steel frames has increased significantly since these were constructed, resulting in a current Code seismic force significantly larger than the design force. A lack of current Code ductile detailing in the braced frames, shear walls, and drag connections may result in greater localized damage; though these effects are expected to be less pronounced in a shorter/stiffer building. Site is susceptible to liquefaction during a seismic event. The existing drawings indicate that some of the older piles do not penetrate the denser sand layer adequately, and it is estimated that those piles may settle several inches in a seismic event. This will cause distress in the old, concrete building framing and floor systems, along with damage to the secondary structural systems. The seismic upgrade work performed in this area utilized micropiles to withstand the seismic forces at the new shear walls. These micropiles are deep enough that they are expected to experience very small settlement, which may lead them to become overloaded where adjacent to older piles that may settle and shed the building loads to them. The slab-on-grade may see liquefaction settlement of one foot or more, which will result in damage to architectural and MEP systems supported by the slab

The evaluation of the existing structure using ASCE 41 performance criteria and site specific response spectra developed for this study, results in the following estimates of return period earthquake for the three performance levels considered (without consideration of liquefaction effects) as shown in Figure 13:

Immediate Occupancy:	140 years
Life Safety:	1400 years
Collapse Prevention:	2500 years

MEP equipment and systems are generally braced and expected to behave satisfactorily for an enhanced life-safety level design, except where affected by liquefaction settlement; however, the systems vary greatly in age and several may remain that were installed prior to typical bracing requirements.

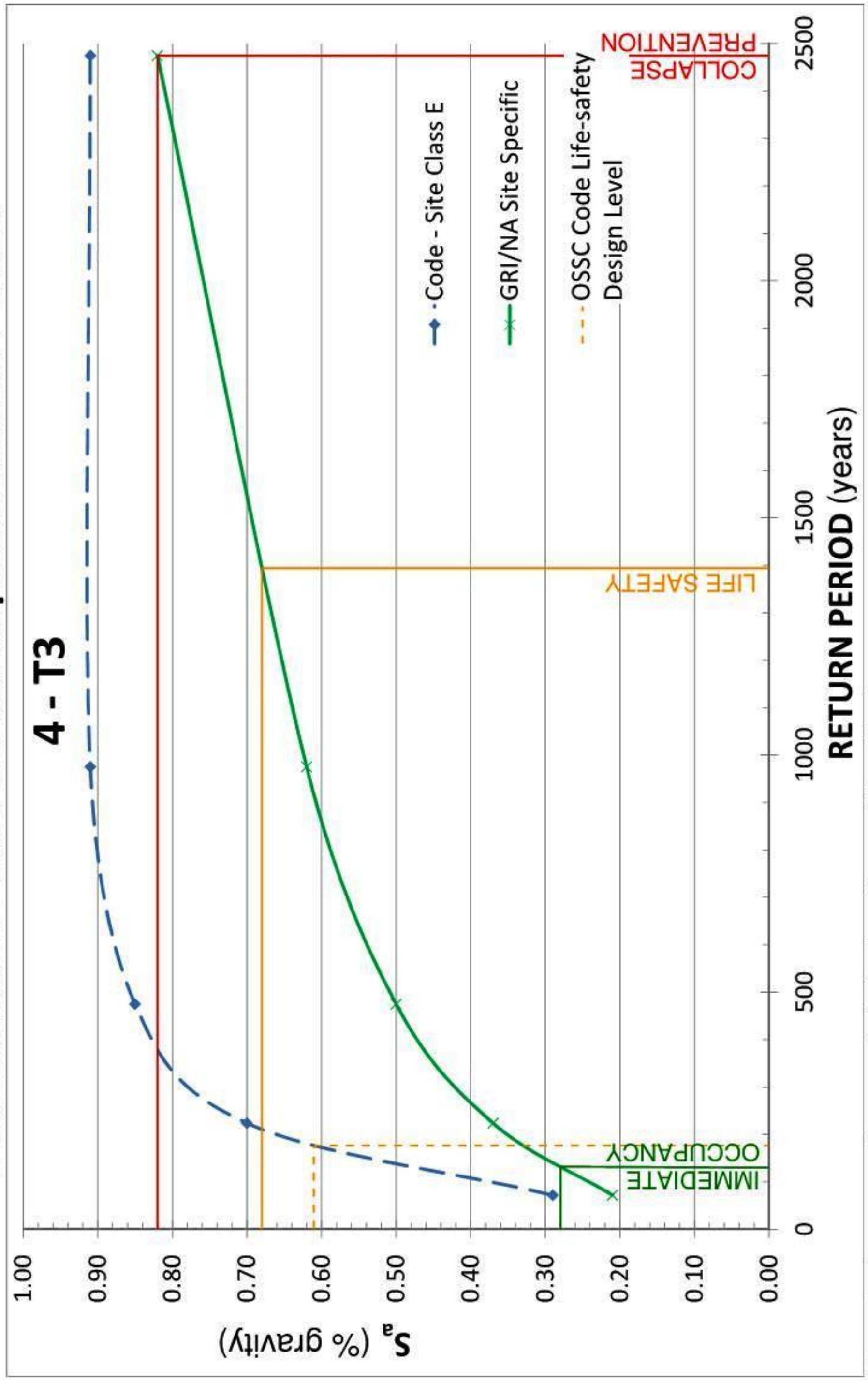
Mitigation: Mitigation will necessarily address both the liquefaction issues along with the seismic strength and ductility of the lateral force resisting system (LFRS). Potential mitigation schemes are identified below. Refer to Figure 14a, 14b and 14c for proposed concept mitigation scheme.

Liquefaction: The settlement of the building columns and structural walls due to settlement of the existing older piles may be addressed by the addition of new micropiles at each column. This would involve slab demolition as well as penetrating the existing pile caps to install the new piles without major disruption of airport functions. These pile caps are identified in the plan following. The slab-on-grade may be mitigated by removing and replacing with a structural slab that spans to the reinforced pile caps, or micropiles under the existing slab-on-grade to reduce the effective span to approximately 10 feet. A combination of these methods may be appropriate, depending on existing constraints. Sequencing slab mitigation may also be possible, for example, mitigating areas supporting critical MEP equipment over open assembly space and/or weighing the risk of postponing the work in the BHS area.

Lateral Force Resisting System: The LFRS consists of a combination of concrete shear walls and steel concentric braced frames. The concrete shear walls are expected to perform adequately for Immediate Occupancy for the 475-year seismic event. The braced frames do not have current Code ductility, and the overall performance of this area could be improved by replacing these with more ductile buckling-restrained braces, which have advantages in ductility performance, along with limiting the force that will be transferred to the columns. Several of the braced frames are inverted-V configurations, and their performance could be improved by the addition of columns in line with the point of the V, or possibly changing the configuration to a single diagonal. Additionally, there is one steel braced frame at the Deplaning level in this area, which is the only braced frame at this level in the Terminal, and one braced frame along grid G, that should be considered to be replaced with concrete shear walls for better compatibility of performance with the other LFRS elements. It should be anticipated that certain drag and diaphragm connections will also require reinforcing. Additional piles to resist seismic overturning forces may be required, as the acceptance criteria for these elements is significantly higher for Immediate Occupancy. This will require consideration, but may in part be in conjunction with adding piles to eliminate gravity column settlements.

Construction Cost Estimate: $20,000,000.

Next Steps: Detailed geotechnical, structural, and MEP assessments to verify the liquefaction effects, site response spectra, and assess any MEP systems that may be critical for Immediate Occupancy. Structural analyses may include more intensive non-linear or time-history non-linear analyses to most accurately evaluate the structure and provide upgrades that achieve the desired performance while optimizing construction and disruption costs.



Note: Performance does not include the effects of liquefaction.

Figure 13

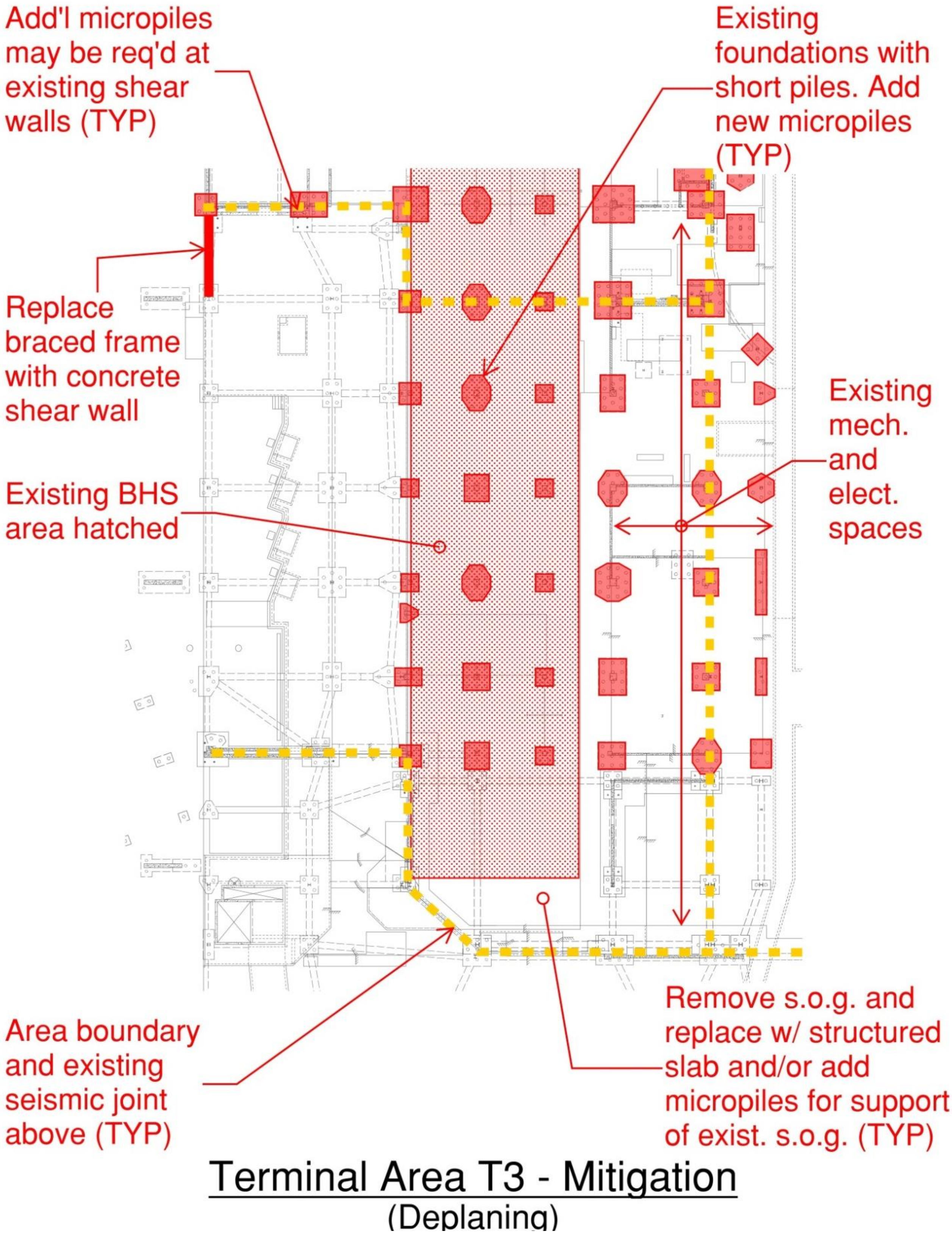

Add'l micropiles may be req'd at existing shear walls (TYP)
Existing foundations with short piles. Add new micropiles (TYP)
Replace braced frame with concrete shear wall
Existing BHS area hatched
Existing mech. and elect. spaces
Area boundary and existing seismic joint above (TYP)
Remove s.o.g. and replace w/ structured slab and/or add micropiles for support of exist. s.o.g. (TYP)
Terminal Area T3 - Mitigation
(Deplaning)

Figure 14a

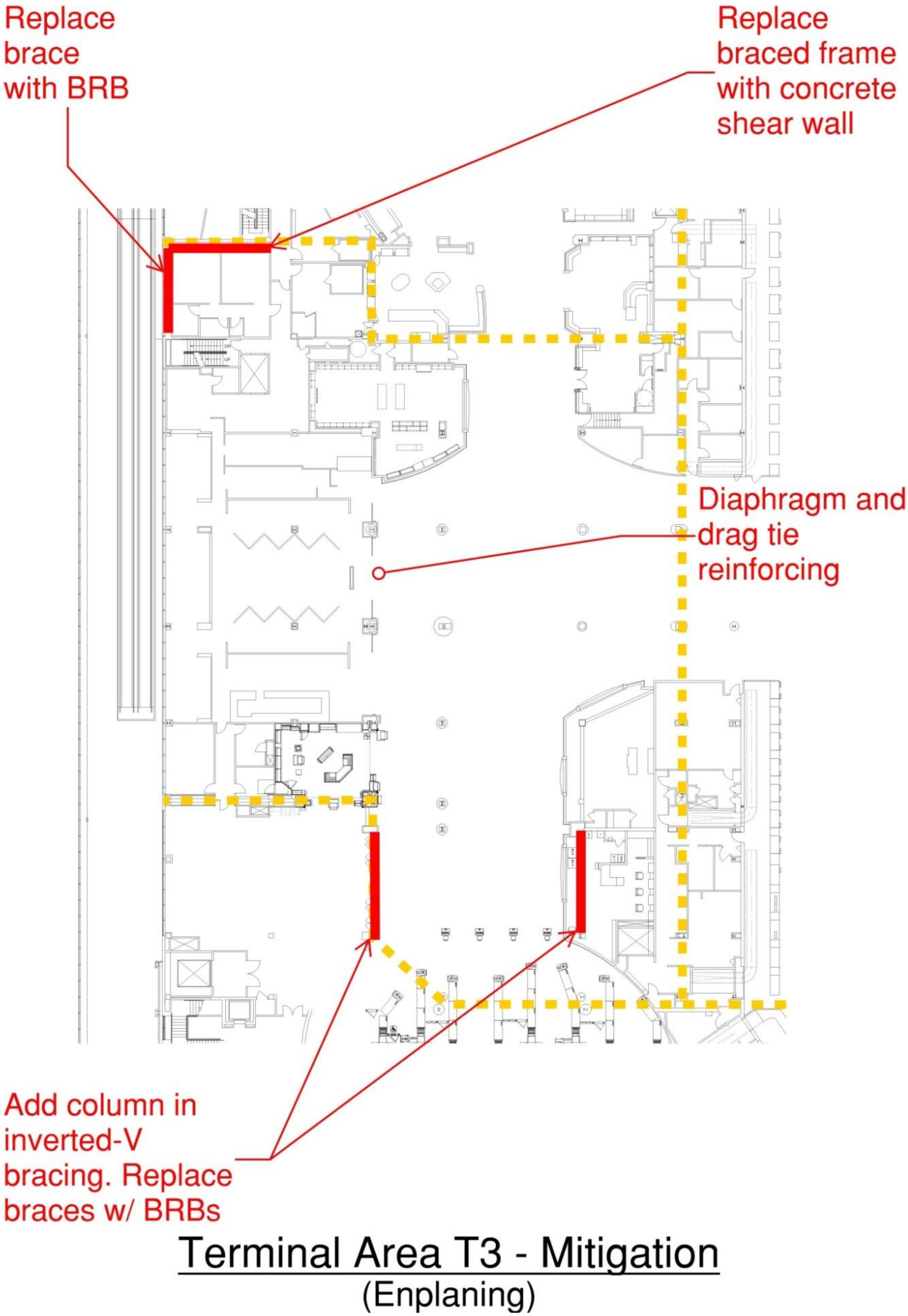

Replace brace with BRB
Replace braced frame with concrete shear wall
Diaphragm and drag tie reinforcing
Add column in inverted-V bracing. Replace braces w/ BRBs
Terminal Area T3 - Mitigation
(Enplaning)

Figure 14b

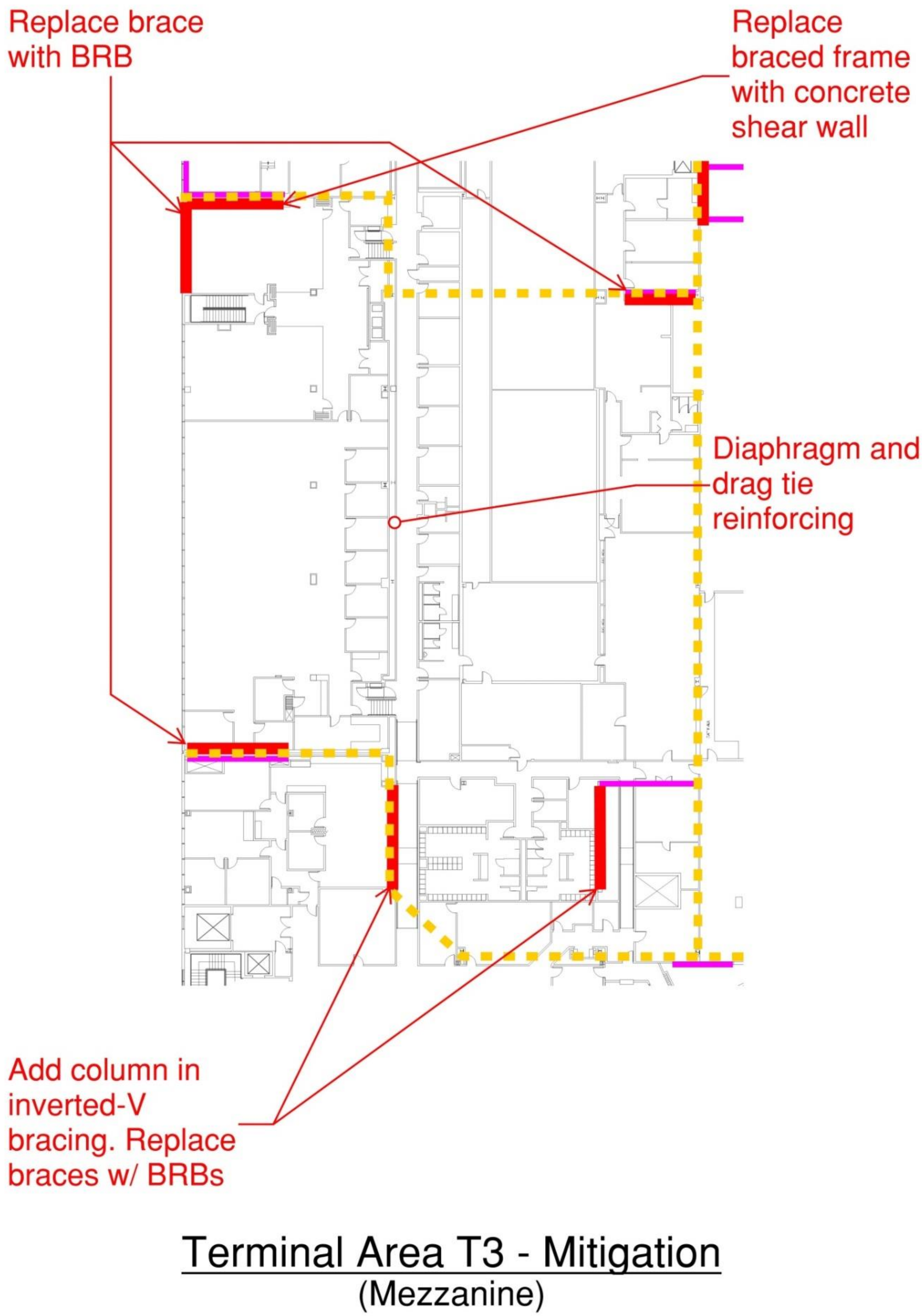
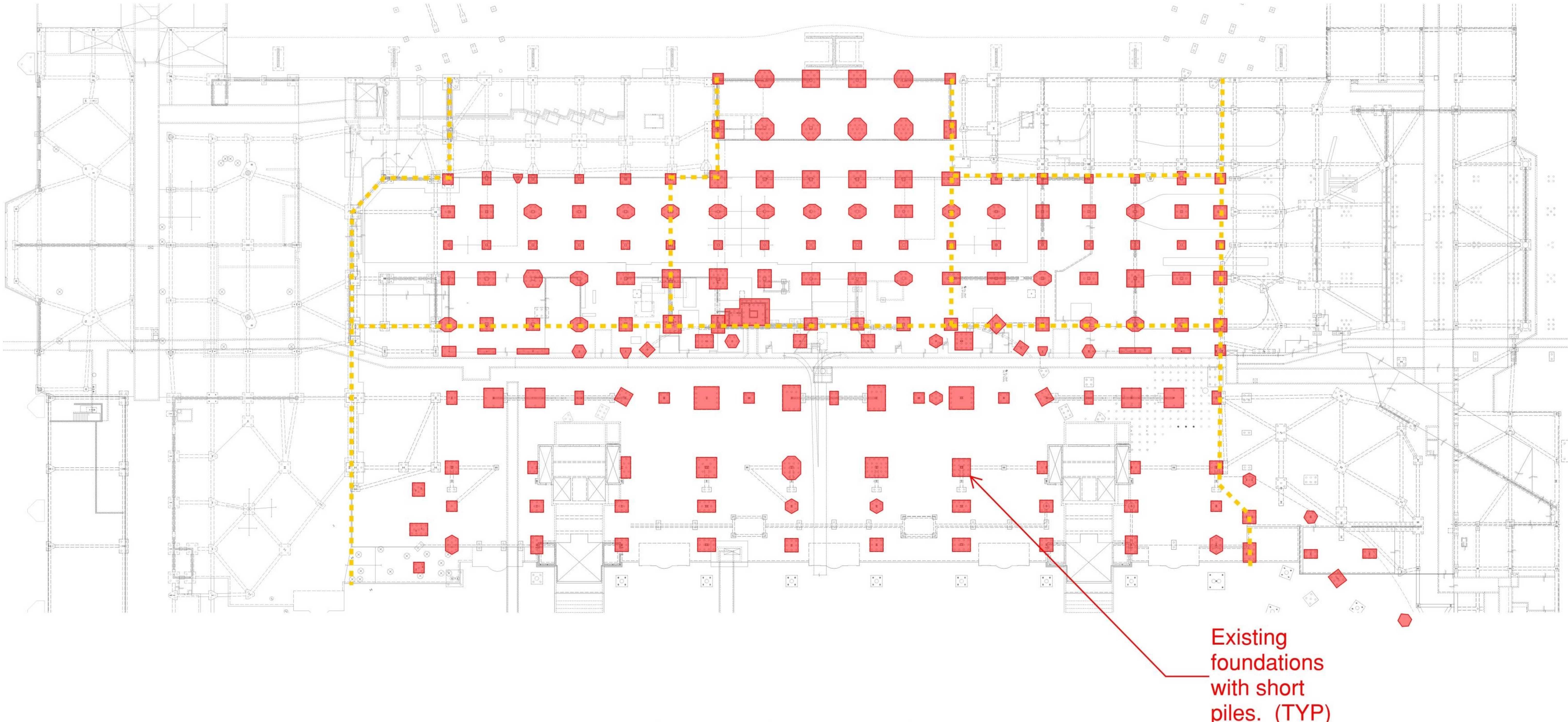

Replace brace with BRB
Replace braced frame with concrete shear wall
Diaphragm and drag tie reinforcing
Add column in inverted-V bracing. Replace braces w/ BRBs
Terminal Area T3 - Mitigation
(Mezzanine)

Figure 14c



Terminal Building Overall Plan

Figure 15

Terminal – Oregon Market Place Central

Building ID#: 4 - T4

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The central portion of the Oregon Market Place is a three-story structure including the mezzanine. It consists of portions built in different construction projects, including the Oregon Market Place project that demolished a portion of the roof structure to add a raised clerestory.

Building Structural System: The structure includes one-way concrete slab and beam construction from 1956, along with a new roof over the western portion constructed in 1986 of steel and composite concrete on metal deck. The 1956 concrete structure lacks current detailing, in particular in the lack of shear reinforcing of the beams. Ground floor is slab-on-grade. Foundations are supported on piles. This portion of the terminal has been part of the ongoing voluntary seismic upgrade program. The most recent elements were installed in the Baggage System Improvements (BSI) project, which completed the upgrade at the Deplaning Level. The upgrade is incomplete on the Enplaning and Mezzanine Levels. The upgrade consists of a combination of steel concentric braced frames and concrete shear walls.

Code Summary: Designed for the 1997 UBC (1998 OSSC) for force level. Detailing met Code at time of construction (1991 UBC to 2007 OSSC).
Base shear design Code = 107% base shear of current Code for concrete shear wall construction.
Base shear design Code = 70% base shear of current Code for steel concentric braced frame construction.
Shear wall reinforcing and steel braced frame detailing is presumed to meet "ordinary" criteria, which is no longer a permitted construction type for new buildings in this seismic zone.

General Seismic Evaluation: Currently, the Terminal-Oregon Market Place Central has an incomplete seismic upgrade. The lateral system is a combination of concrete shear walls and concentric braced frames, the older ones of which lack current Code ductile detailing requirements. The required seismic design force for ordinary steel frames has increased significantly since these were constructed, resulting in a current Code seismic force significantly larger than the design force. The expected performance of this section will be poor, until the seismic upgrade is completed. If the upgrade is completed without addressing the existing lower-ductility elements, the performance for the basic Code performance objective of Life-safety at 2/3 of the acceleration of the 2475-year return period earthquake, may be marginal. Based on site-specific spectra developed in this study, Collapse-prevention performance may only be achieved for approximately 1000-year return period event. Consideration should be given to addressing the ductility of the system as a whole when completing the seismic upgrade to improve this performance.

The site is subject to liquefaction. It appears that the piles in this original construction do not penetrate the medium dense to dense sands adequately, and may experience settlements of several inches in an earthquake. The expected differential settlements can be anticipated to result in substantial damage to the older concrete structure; likely requiring extensive repair or possibly rebuilding. Additionally, the slab-on-grade is expected to experience settlements of approximately 10" under a 500-year return period event, and over one foot for larger earthquakes. These settlements will impact functions and services that are supported by the slab, including baggage and MEP systems

MEP equipment and systems are generally braced and expected to behave satisfactorily for a life-safety level design; however, the systems vary greatly in age and several may remain that were installed prior to typical bracing requirements.

Next Steps: If this area is to remain intact in the long-term plans, construction of the remaining portions of the voluntary seismic upgrade should be the first priority. Note that completing the upgrade may be required if any substantial work is performed in the area, regardless of long-range plans, unless other arrangements are made with the City of Portland. Prior to construction, consideration should be given as to the desired level of performance after completion of the upgrades. This may involve completing the basic upgrade with the intent of "enhanced" upgrades at a later time to achieve the desired performance if access to certain areas are not practical or economical at that time. Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. A detailed geotechnical investigation should be considered prior to this work to substantiate, or adjust, the recommendations found in this report. If performance levels above Life Safety are desired, a MEP review of critical systems and their resilience should also be included. Further structural analyses may include more intensive non-linear or time-history non-linear analyses to most accurately evaluate the structure and provide upgrades that achieve the desired performance while optimizing economy.

Aircraft Rescue and Firefighting Facility (ARFF)

Building ID#: 5

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The Fire Station is a single-story structure with two partial mezzanines and pitched roofs. The western portion houses the fire and rescue trucks.

Building Structural System: The building is constructed with concrete masonry bearing/shear walls. The roof over the eastern portion is composite concrete on metal deck, with metal roof deck over the remainder. One mezzanine is constructed of steel with composite concrete on metal deck and one is wood framed. The foundation is a mat slab consisting of 12” slab-on-grade, thickened to 18” to 24” under the bearing walls. The lateral system is masonry shear walls with combination steel moment frame/masonry shear wall piers at the truck bays.

Code Summary: Constructed under the 1991 UBC (1993 OSSC) as an Essential Facility with an Importance Factor = 1.25.
Base shear design Code = 107% base shear of current Code.

Masonry shear walls appear to meet “intermediate shear wall” criteria. Current Code requires “special” reinforcing criteria for new buildings in this seismic zone.

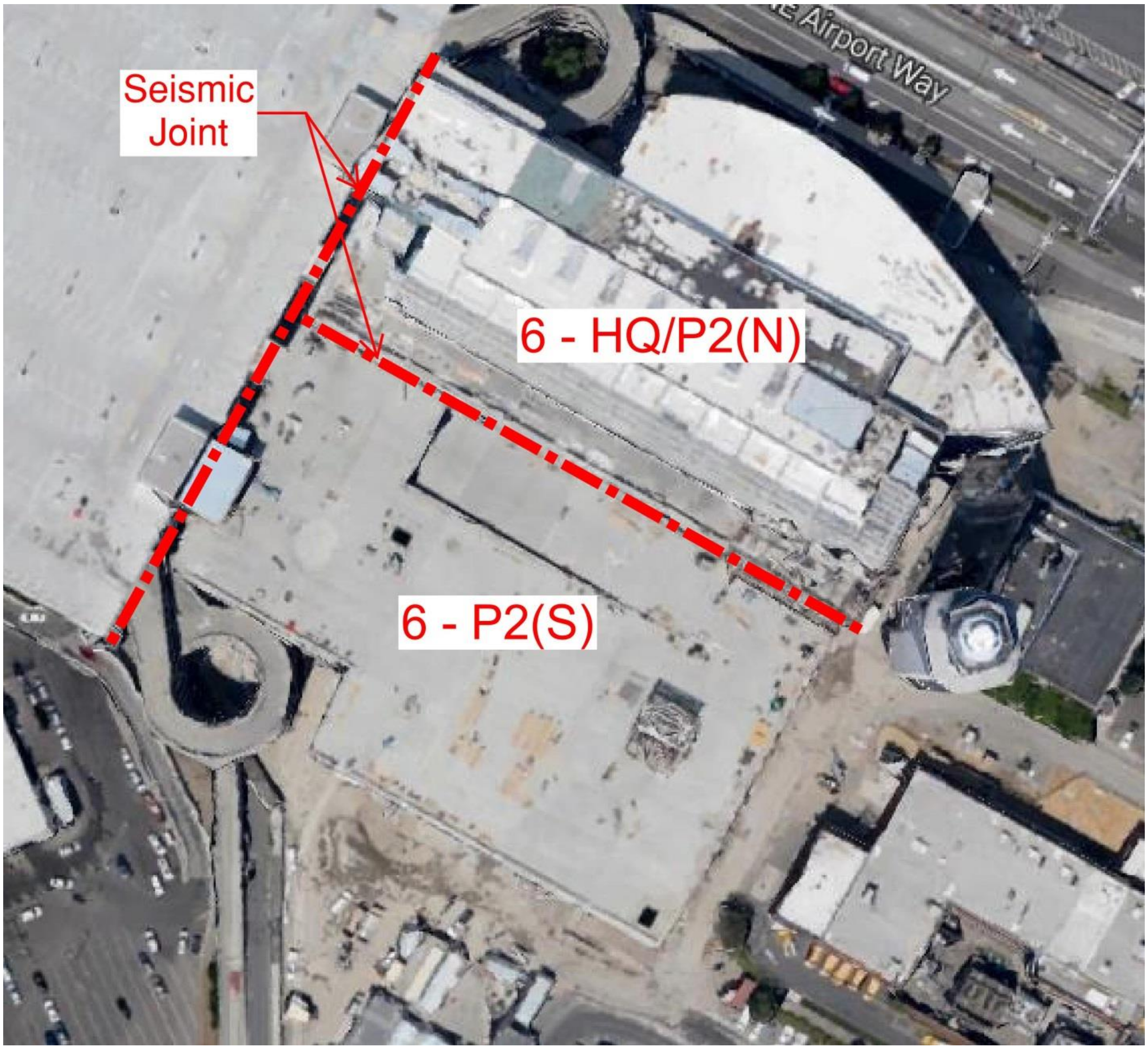
General Seismic Evaluation: The Aircraft Rescue and Firefighting Facility meets the current Code seismic design force for an Essential Facility in Seismic Design Category D. However, it may lack some of the newer steel reinforcing ductility requirements of current Code. A lack of ductile detailing may result in greater localized damage; though these effects are expected to be less pronounced in a short, stiff building. The original design was based on a limited liquefaction analysis consistent with geotechnical knowledge in the industry at the time of design. Minimal liquefaction settlement was expected with some potential for lateral spread. Estimates of this study are for minimal lateral spreading, but settlements of approximately six inches at a 200-year return earthquake, and more than one foot for 1000-year and greater earthquakes. The mat foundation will mitigate the effects of settlement to a degree; however, differential settlements may create issues with truck-bay door operations and offsets in interior to exterior slabs.

MEP equipment and systems are generally braced and expected to behave satisfactorily for an essential facility level design. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: A detailed geotechnical, structural, and MEP investigation would give a better understanding of specific risks and potential mitigation strategies. Prior to any mitigation efforts, developing a strategy for extricating the trucks from the structure should be developed, to ensure that necessary tools are available and personnel are aware of their expected roles and actions.

HQ/P2

HQ/P2 Layout:



HQ/P2 North

Building ID#: 6a

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The P2 parking garage has seven levels of parking. This north half includes three levels of office space above (HQ).

Building Structural System: The gravity system for the parking levels consists of post-tensioned concrete beams and floor slabs. The office levels above are steel framing with composite concrete over metal deck slabs. The garage lateral system in the N-S direction is a dual-system, special concrete shear walls with special concrete moment resisting frames, and special moment resisting frames in the E-W direction. The steel framed office levels are special steel moment resisting frames in both directions. The ground floor is slab-on-grade. The foundations are supported on steel piles.

Code Summary: Constructed under the 2006 IBC (2007 OSSC).
Base shear design Code = 100% of base shear of current Code.

General Seismic Performance: HQ/P2 North meets the current Code design and detailing requirements for standard parking and office space in Seismic Design Category D. Code performance expectations are for collapse prevention subjected to a 2475-year return period earthquake, and life-safety under an earthquake with accelerations of 2/3 that of the 2475-year event.

Geotechnical analysis in this study indicates that the life-safety event would correspond to an approximately 900-year return period event. The site is susceptible to liquefaction during a seismic event, which may result in localized settlement of the slab-on-grade. Magnitudes of settlement are expected to exceed a foot with a 1000-year, or greater, return period event. This will cause disruption of functions and services that rely on the slab-on-grade for support.

MEP equipment and systems are braced per current Code and expected to behave accordingly.

Next Steps: A detailed geotechnical, structural, MEP and architectural investigation would give a better understanding of what level of earthquake this building and the critical MEP systems can perform at an Immediate Occupancy level, as well as the drifts that the architectural systems can accommodate. Liquefaction will affect the ground floor and mitigation options will be similar to those discussed for areas of the terminal.

P2 South

Building ID#: 6b

Building Address: 7000 NE Airport Way, Portland, OR 97218

Building Description: The south half of the P2 parking garage has seven levels of parking.

Building Structural System: The gravity system for the parking levels consists of post-tensioned concrete beams and floor slabs. The lateral system in the N-S direction is a dual-system, special concrete shear walls with special concrete moment resisting frames, and special moment resisting frames in the E-W direction. The ground floor is slab-on-grade. The foundations are supported on steel piles. The building was designed to accommodate three levels of office space similar to P2 North.

Code Summary: Constructed under the 2006 IBC (2007 OSSC).
Base shear design Code = 100% of base shear of current Code.

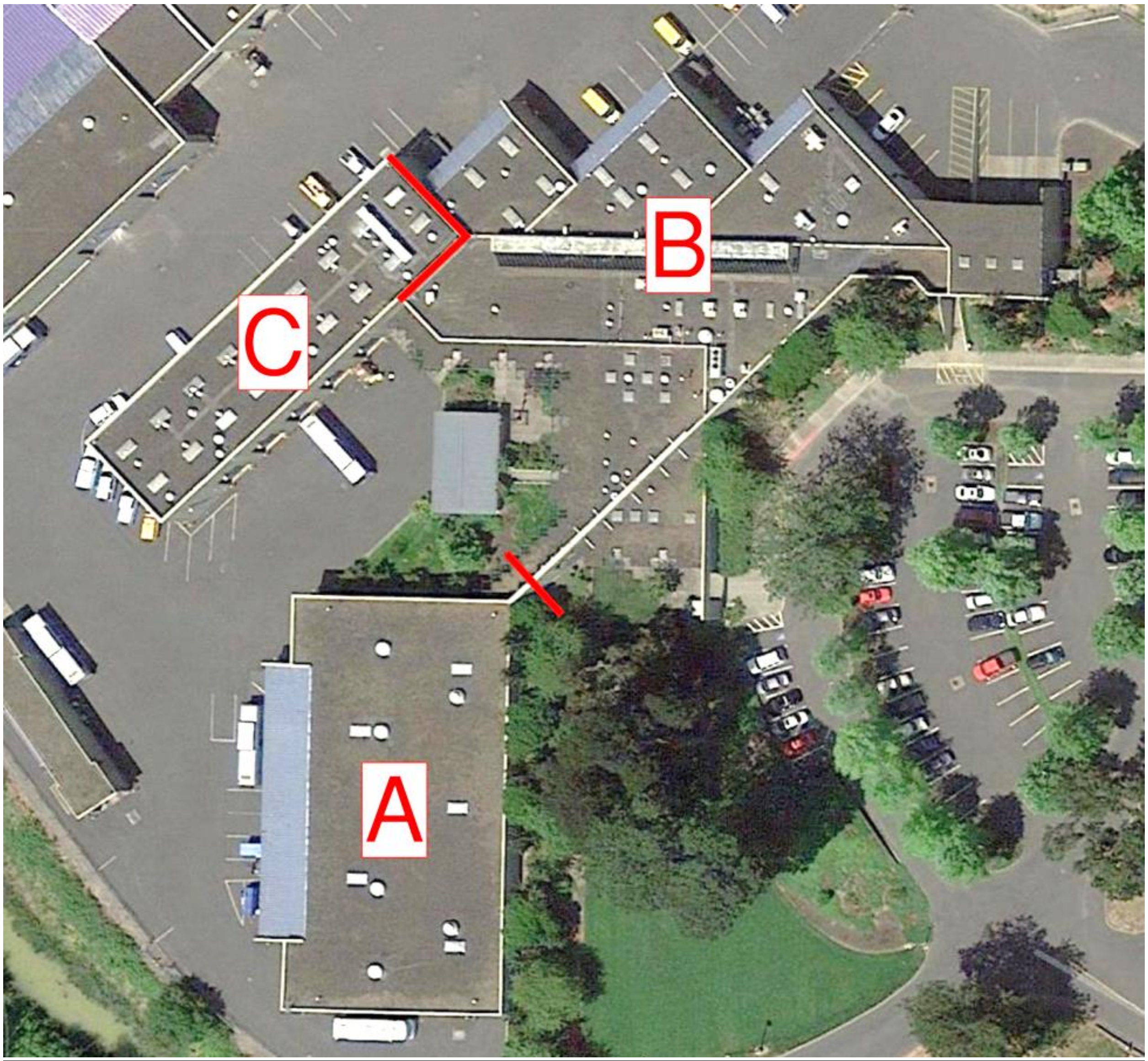
General Seismic Performance: P2 South meets the current Code design and detailing requirements for standard parking in Seismic Design Category D. Without the office addition, the existing P2 structure exceeds the Code seismic design force requirement for performance expectations of collapse prevention under a 2475-year return period earthquake, and life-safety under an earthquake with accelerations of 2/3 that of the 2475-year event. The site is susceptible to liquefaction during a seismic event, which may cause localized settlement in the slab-on-grade. Magnitudes of settlement are expected to exceed a foot with a 1000-year, or greater, return period event. The utility tunnel from the Central Utility Plant runs under P2 South, and connects with the pedestrian tunnel. This will cause disruption of functions and services that rely on the slab-on-grade for support. The tunnels are pile-supported and should not experience direct impacts from the liquefaction.

MEP equipment and systems are braced per current Code and expected to behave accordingly.

Next Steps: A detailed geotechnical, structural, and MEP investigation would give a better understanding of what level of earthquake this building can perform at an Immediate Occupancy level. Liquefaction will affect the ground floor and mitigation options will be similar to those discussed for areas of the terminal.

Ground Maintenance Administration and Shops (Buildings B, A and C)

Ground Maintenance Administration and Shops Layout:



Ground Maintenance Administration and Shops (Building B)

Building ID#: 13

Building Address: 7111 NE Alderwood Road, Portland, OR 97218

Building Description: Building B is a one-story structure comprised of office, storage and maintenance spaces, with a small second story section with a balcony and two interior mezzanines. The south side of the building has a large soil berm against the building face up to mid-height of the structure.

Building Structural System: Precast, hollow-core, concrete panels perform as exterior bearing/shear walls, supporting a majority of both the gravity and lateral loads of the structure. The precast panels along the south building face are supported by a cast-in-place concrete retaining wall, due to the soil berm. Interior steel framing supports roof members, as well as the two mezzanines. The roof structure is comprised of either wood framing with plywood diaphragms spanning up to 140 feet, or open web three-dimensional “Unistrut” trusses with steel roof decking.

Code Summary: Constructed under the 1979 UBC (1980 OSSC).

Base shear design Code = 31% of current Code. Precast wall panels are presumed to meet “ordinary precast shear walls” detailing criteria, and masonry walls are presumed to meet “ordinary reinforced masonry shear walls” detailing criteria. “Ordinary” shear wall systems are no longer a permitted construction type for new buildings in this seismic zone.

General Seismic Performance: The Ground Maintenance Administration and Shops was constructed prior to the significant increases in seismic design loads that occurred in the 1990’s, and has a lateral system that has been penalized by Code due to a lack of ductility. The roof diaphragm is lacking continuous cross-ties and the ledger is loaded in cross-grain bending which are common failure mechanisms in buildings of this type and era. Additionally, the roof levels in Buildings B and C are offset, which can cause significant damage to the structure in that area. One of the interior mezzanines has steel concentric braced frames; but the other appears to be unbraced. The site is susceptible to liquefaction, which may cause ground settlements of 1½ feet under earthquakes with a return period as short as 200 years. Spread footings may see an additional foot of settlement. Settlements of this magnitude, along with the other seismic deficiencies, are expected to result in a building that is unusable after an approximately 200-year return period, or greater, earthquake.

MEP equipment and systems appear to be unbraced, but are mostly of a relatively minor nature. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. It is quite likely that the cost of such mitigation will make replacement of the facility a preferable option; particularly when viewed with the superior performance (structurally, functionally, and energy efficiency) of a new building. A detailed geotechnical investigation could yield more accurate liquefaction predictions; however, it is unlikely that the effects would be found to be negligible.

Ground Maintenance Facility (Building A)

Building ID#: 14

Building Name: Ground Maintenance Facility (Building A)

Building Address: 7111 NE Alderwood Road, Portland, OR 97218

Building Description: Building A is a large one-story vehicle parking and storage warehouse. The east side of the building has a large soil berm against the building face up to mid-height of the structure.

Building Structural System: Precast, hollow-core, concrete panels perform as exterior bearing/shear walls, with interior steel columns supporting glulam beams and open-web, steel joists at the roof level. The roof diaphragm is plywood sheathing, spanning 180 feet between shear walls. The precast panels along the east building face are supported by a cast-in-place concrete retaining wall that resists the pressure of the soil berm. The ground floor is slab-on-grade. The foundations are spread footings.

Code Summary: Constructed under the 1979 UBC (1980 OSSC).
Base shear design Code = 37% of current Code. Precast wall panels are presumed to meet "ordinary precast shear walls" detailing criteria, which is no longer a permitted construction type for new buildings in this seismic zone.

General Seismic Performance: The Ground Maintenance Facility was constructed prior to the significant increases in seismic design loads that occurred in the 1990's, and has a lateral system that has been penalized by Code due to a lack of ductility. The roof diaphragm is lacking continuous cross-ties and the ledger is loaded in cross-grain bending which are common failure mechanisms in buildings of this type and era. The site is susceptible to liquefaction, which may cause ground settlements of 1½ feet under earthquakes with a return period as short as 200 years. Spread footings may see an additional foot of settlement. Settlements of this magnitude, along with the other seismic deficiencies, are expected to result in a building that is unusable after an approximately 200-year return period, or greater, earthquake.

MEP equipment and systems are of a relatively minor nature in this building. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. It is quite likely that the cost of such mitigation will make replacement of the facility a preferable option; particularly when viewed with the superior performance (structurally, functionally, and energy efficiency) of a new building. A detailed geotechnical investigation could yield more accurate liquefaction predictions; however, it is unlikely that the effects would be found to be negligible.

<u>Ground Maintenance Facility (Building C)</u>

Building ID#: 15

Building Address: 7111 NE Alderwood Road, Portland, OR 97218

Building Description: Building C is a one-story vehicle maintenance and supply storage warehouse with a small interior mezzanine.

Building Structural System: Precast, hollow-core, concrete panels perform as exterior bearing/shear walls, with interior steel columns supporting glulam beams and solid wood joists at the roof level. The roof diaphragm is plywood sheathing, spanning up to 140 feet between shear walls. The mezzanine has CMU walls with a reinforced concrete floor slab. The ground floor is slab-on-grade. The foundations are spread footings.

Code Summary: Constructed under the 1979 UBC (1980 OSSC).
Base shear design Code = 31% of current Code in the N-S direction, (a combination of precast panels and masonry walls) and 37% in the E-W direction (precast panels only). Precast wall panels are presumed to meet "ordinary precast shear walls" detailing criteria, and masonry walls are presumed to meet

"ordinary reinforced masonry shear walls" detailing criteria. These systems are no longer permitted construction types for new buildings in this seismic zone.

General Seismic Performance: The Ground Maintenance Facility was constructed prior to the significant increases in seismic design loads that occurred in the 1990's, and has a lateral system that has been penalized by Code due to a lack of ductility. The roof diaphragm is lacking continuous cross-ties and the ledger is loaded in cross-grain bending which are common failure mechanisms in buildings of this type and era. Additionally, the roof levels in Buildings B and C are offset – 18 feet vs. 27 feet, which can cause significant damage to the structure in that area. The site is susceptible to liquefaction, which may cause ground settlements of 1½ feet under earthquakes with a return period as short as 200 years. Spread footings may see an additional foot of settlement. Settlements of this magnitude, along with the other seismic deficiencies, are expected to result in a building that is unusable after an approximately 200-year return period, or greater, earthquake.

MEP equipment and systems appear to be unbraced, but are mostly of a relatively minor nature. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. It is quite likely that the cost of such mitigation will make replacement of the facility a preferable option; particularly when viewed with the superior performance (structurally, functionally, and energy efficiency) of a new building. A detailed geotechnical investigation could yield more accurate liquefaction predictions; however, it is unlikely that the effects would be found to be negligible.

Maintenance Warehouse at T-6

Building ID#: 16

Building Address: 7205 N Marine Drive, Portland, OR 97203

Building Description: The Maintenance Warehouse is a Butler-type, pre-fabricated steel building. It consists of a one-story warehouse/shop space with two mezzanines, as well as a two-story office space.

Building Structural System: The building consists of steel framing with precast concrete wall panels bolted to the frames extending to mid-height of the structure. The roof is framed with z-girts and metal roof deck. The upper side walls are framed with Z-girts and metal panels. There are two interior mezzanines with reinforced masonry walls, as well as a two-story infill portion consisting of full height reinforced masonry walls up to the roof level. The framing of the mezzanines and 2nd story are open web steel joists supporting concrete topping over metal decking. The lateral system is comprised of two bays of tension-only steel rod bracing occuring along each exterior building wall, from the top of the precast panels to the roof diaphragm, along with moment frames in the east-west direction. The ground floor is slab-on-grade. Foundations are spread footings.

Code Summary: Constructed under the 1970 UBC. Original design was wind-based.
Design wind load = 77% of seismic design load of the current Code in the east-west direction, and 35% of the seismic design load of the current Code in the north-south direction.
Steel frames presumed to meet “ordinary moment frame” detailing criteria in the E-W direction. The N-S direction has tension-only rod bracing in combination with precast concrete wall panels presumed to meet “ordinary precast shear wall” detailing criteria. Tension-only bracing is no longer a permitted construction type for new buildings in this seismic zone.

General Seismic Performance: The current Code design forces exceed the original design forces for the Maintenance Building, particularly in the north-south direction. The rod bracing and precast panels in the north-south direction lack the ductile configuration and detailing requirements of current Code requirements. The moment frames in the east-west direction also lack current Code ductility, although would be expected to perform better than the tension-rod bracing. Additionally, the site is susceptible to liquefaction during a seismic event, exceeding one foot in settlement at relatively short return periods (less than 300 years). The structure and foundations are not designed to accommodate this magnitude of settlement, so extensive damage would be anticipated from the liquefaction alone.

MEP equipment and systems appear to be unbraced, but are mostly of a relatively minor nature. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. A more detailed geotechnical, structural, and MEP investigation could determine what mitigation strategies, if any, might be practical and economical for a building of this type and considering its remaining useful service life.

<u>Electrical Shop at T-6</u>

Building ID#: 17

Building Address: 7209 N Marine Drive, Portland, OR 97203

Building Description: The Electrical Shop is a Butler-type, pre-fabricated building. It consists of a one story warehouse/shop space with a mezzanine, as well as a two-story office space.

Building Structural System: The Shop is a steel framed building with roof and wall framing consisting of steel Z-girts and metal panels. Ground floor is slab-on-grade. Foundations are spread footings. The lateral system is comprised of steel moment frames in the east-west direction and two observable bays of tension-only cable bracing on each side of the structure in the north-south direction.

Code Summary: Constructed under the 1985 UBC (1986 OSSC). Original design was wind-based.

Design wind load = 167% of seismic design load of the current Code in the east-west direction, and 28% of the seismic design load of current Code in the north-south direction.

Steel frames presumed to meet “ordinary moment frame” detailing criteria in the E-W direction. The N-S direction has tension-only rod bracing in which is no longer a permitted construction type for new buildings in this seismic zone.

General Seismic Performance: The original wind load exceeds the current Code design force for the Electrical Building in the east-west direction; however the design wind load is much less than the current Code seismic force in the north-south direction. The rod bracing in the north-south direction also lacks the ductile configuration and detailing requirements of current Code. The moment frames in the east-west direction also lack current Code ductility, but appear to have a significant excess capacity relative to design force. The full-height concrete masonry walls do not appear to proper attachments to, or isolation from, the roof and walls, which may result in damage and falling debris hazards. Additionally, the site is susceptible to liquefaction during a seismic event, exceeding one foot in settlement at relatively short return periods (less than 300 years). The structure and foundations are not designed to accommodate this magnitude of settlement, so extensive damage would be anticipated from the liquefaction alone.

There were a number of storage cabinets and parts racks that were not seismically braced which pose a hazard during a seismic event.

MEP equipment and systems are generally unbraced, but of a minor nature. However, with the liquefaction settlements, underground utilities and their connections to the building-supported utilities may be compromised.

Next Steps: Mitigation would need to address both the seismic deficiencies as well as the liquefaction issues. A more detailed geotechnical, structural, and MEP investigation could determine what mitigation strategies, if any, might be practical and economical for a building of this type and considering its remaining useful service life.

SUPPLEMENT

Definitions

ASCE 41

American Society of Civil Engineers - *Seismic Evaluation and Retrofit of Existing Buildings*.

Buckling-restrained Brace (BRB)

A brace system in which a force-resisting steel core in debonded from a larger casing that resists no load, but prevents the buckling of the core. Compared to typical braces, a BRB exhibits greater ductility and equal capacities and effective stiffness in tension and compression.

Ductility

Capacity of a system to undergo cycles of stress with deformation; but not fracture.

Fluid Viscous Damper (FVD)

An element that uses the viscosity of a fluid to resist movement and increase the effective damping of a system, installed as part of a structural lateral bracing system. Resistance is proportional to velocity, as opposed to acceleration, so that induced forces are out of phase with typical bracing systems.

Force Capacity

Strength of the system to resist the seismic forces for the performance level under consideration, without consideration of ductility.

IBC

International Building Code.

Lateral Force Resisting System (LFRS)

The elements of the building system that provide the required resistance to prescribed seismic forces, such as shear walls, braced frames, and moment frames.

Liquefaction

Loss of strength and stiffness of saturated soil during shaking.

MEP

Mechanical, Electrical, Plumbing and similar infrastructure systems.

Micropile

A pile system consisting of a small diameter pipe casing (approximately 8”), with a center reinforcing bar extending into a grout zone beyond the bottom of the casing. May be installed with small equipment in relatively tight areas.

OSSC

Oregon Structural Specialty Code.

Performance Objective

Acceptable, or desired, damage and functionality level with respect to a given seismic event.

Primary Structural System

The main building structure supporting gravity loads and overall building stability, such as beams, columns, slabs, foundations, and walls, and including the LFRS.

Response Spectra

A plot of the seismic acceleration of a building versus building oscillation frequency.

Secondary Structural System

The structural systems other than the Primary Structural System, such as the building envelope (skin), glazing, stairs, and MEP systems.

Sa

Spectral response acceleration.

Ss

Short period spectral response acceleration.

UBC

Uniform Building Code.

Opinion of Probable Construction Costs

BID ITEM	APPROX. QUANTITY	UNIT	UNIT COST ($/UNIT)	TOTAL COST ($)
1 - CUP				
New Concrete Shear Walls	7	EA	$60,000	$420,000
Micropiles - Building	70	EA	$20,000	$1,400,000
Micropiles - S.O.G. and Yard Equipment	60	EA	$20,000	$1,200,000
Exterior Wall Repair or Replacement	16,000	SF	$50	$800,000
Drag ties/Diaphragms (Mezz)	10,000	SF	$10	$100,000
Drag ties/Diaphragms (Roof)	21,000	SF	$10	$210,000
MEP	31,000	SF	$5	$155,000
MEP Perimeter Connections	1	LS	$200,000	$200,000
Tunnel	80	LF	$4,000	$320,000
Subtotal				**$4,805,000**
Direct Costs			100.0%	$4,805,000
Indirect Costs			30.0%	$2,883,000
Contingency			30.0%	$3,747,900
TOTAL				**$16,240,900**
TOTAL PER SF				**$524**
3 - C1 CONCOURSE C - EAST				
Fluid Viscous Dampers	26	EA	$30,000	$780,000
Micropiles - Tunnel	10	EA	$20,000	$200,000
Drag ties/Diaphragms (Roof)	37,000	SF	$5	$185,000
Drag ties/Diaphragms (Enpl)	37,000	SF	$3	$111,000
Slab on Grade - Composite Topping Slab	37,000	SF	$15	$555,000
Slab on Grade - Micropiles	100	EA	$20,000	$2,000,000
MEP	74,000	SF	$3	$222,000
Subtotal				**$4,053,000**
Direct Costs			100.0%	$4,053,000
Indirect Costs			30.0%	$2,431,800
Contingency			30.0%	$3,161,340
TOTAL				**$13,699,140**
TOTAL PER SF				**$185**
3 - C2 CONCOURSE C - MIDDLE				
Fluid Viscous Dampers	46	EA	$30,000	$1,380,000
Micropiles - Tunnel	18	EA	$20,000	$360,000
Drag ties/Diaphragms (Roof)	65,000	SF	$5	$325,000
Drag ties/Diaphragms (Enpl)	65,000	SF	$3	$195,000
Slab-on-grade Mitigation	65,000	SF	$100	$6,500,000
MEP	130,000	SF	$3	$390,000
Subtotal				**$9,150,000**
Direct Costs			100.0%	$9,150,000
Indirect Costs			30.0%	$5,490,000
Contingency			30.0%	$7,137,000
TOTAL				**$30,927,000**
TOTAL PER SF				**$238**
3 - C3 CONCOURSE C - WEST				
Fluid Viscous Dampers	54	EA	$30,000	$1,620,000
Micropiles - Tunnel	20	EA	$20,000	$400,000
Drag ties/Diaphragms (Roof)	77,000	SF	$5	$385,000
Drag ties/Diaphragms (Enpl)	77,000	SF	$3	$231,000
Slab-on-grade Mitigation	77,000	SF	$100	$7,700,000
MEP	154,000	SF	$3	$462,000
Subtotal				**$10,798,000**
Direct Costs			100.0%	$10,798,000
Indirect Costs			30.0%	$6,478,800
Contingency			30.0%	$8,422,440
TOTAL				**$36,497,240**
TOTAL PER SF				**$237**
CONC C TOTAL				**$81,123,380**

Opinion of Probable Construction Costs

4 - T1 Terminal Ticket Lobby				
BRB Frame Replacement	6	EA	$20,000	$120,000
Mezzanine Minor Brace Retrofits	8	EA	$10,000	$80,000
Micropiles - Building	160	EA	$20,000	$3,200,000
Micropiles - Tunnel	20	EA	$20,000	$400,000
Drag ties/Diaphragms (Roof)	53,000	SF	$5	$265,000
Drag ties/Diaphragms (Mezz)	10,000	SF	$10	$100,000
Drag ties/Diaphragms (Enpl)	86,000	SF	$5	$430,000
Slab-on-grade Mitigation	86,000	SF	$100	$8,600,000
MEP	149,000	SF	$5	$745,000
Subtotal				**$13,940,000**
Direct Costs			100.0%	$13,940,000
Indirect Costs			30.0%	$8,364,000
Contingency			30.0%	$10,873,200
TOTAL				**$47,117,200**
TOTAL PER SF				**$259**
4 - T2 Terminal South Node				
Micropiles - shear walls	24	EA	$20,000	$480,000
Micropiles - tunnel	22	EA	$20,000	$440,000
Slab-on-grade Mitigation	85,000	SF	$100	$8,500,000
Drag ties/Diaphragms/Boundary Elements (Roof)	80,000	SF	$3	$240,000
Drag ties/Diaphragms/Boundary Elements (Mezz)	21,000	SF	$3	$63,000
Drag ties/Diaphragms/Boundary Elements (Enpl)	93,000	SF	$3	$279,000
MEP	194,000	SF	$3	$582,000
Subtotal				**$10,584,000**
Direct Costs			100.0%	$10,584,000
Indirect Costs			30.0%	$6,350,400
Contingency			30.0%	$8,255,520
TOTAL				**$35,773,920**
TOTAL PER SF				**$180**
4 - T3 Terminal O.M.P. South				
BRB Frame Replacement	14	EA	$20,000	$280,000
RCSW Frame Replacement	800	SF	$100	$80,000
Micropiles	66	EA	$20,000	$1,320,000
Drag ties/Diaphragms (Roof)	30,000	SF	$15	$450,000
Drag ties/Diaphragms (Mezz)	28,000	SF	$10	$280,000
Drag ties/Diaphragms (Enpl)	30,000	SF	$5	$150,000
Slab-on-grade Mitigation	30,000	SF	$100	$3,000,000
MEP	88,000	SF	$5	$440,000
Subtotal				**$6,000,000**
Direct Costs			100.0%	$6,000,000
Indirect Costs			30.0%	$3,600,000
Contingency			30.0%	$4,680,000
TOTAL				**$20,280,000**
TOTAL PER SF				**$230**
Mitigation Total				**$200,535,400**

Notes:
Escalation not included

Direct costs include: logistics and labor factor, phasing, general conditions, O&P, estimating contingency, insurance

Indirect costs include: project management, construction manager and staff, AE design services, AE construction administration, misc.

ORIGINAL DESIGN CODE SEISMIC PARAMETERS

				ORIGINAL DESIGN CODE									CURRENT DESIGN CODE					
Asset	Code	Building Occ.	Lateral System	"Zone"	Z or S_{ds}	I	C or C_a	C*S	R or R_w	K	Design Seismic Base Shear Factor	ASD or USD	S_{ds}	R	I	Current USD Seismic Base Shear Factor	Load Factor	% Current Base Shear Capacity
1- CUP (EW)	1994	EF	OSCBF/ordinary tension-only braced frames	3	0.3	1.25	2.75		8		0.129	ASD	0.61	3.25	1.5	0.282	0.70	65%
1 - CUP (NS)	1994	EF	OSMRF/Ordinary tension-only braced frames	3	0.3	1.25	2.75		6		0.172	ASD	0.61	3.25	1.5	0.282	0.70	87%
3 C1, C2, C3*	1994	Std	SMRF	3	0.3	1	2.75		12		0.069	ASD	0.61	8	1.25	0.095	0.70	103%
4 - Terminal T1	1991	"Non-Ess."	OSCBF	3	0.3	1	2.75		8		0.103	ASD	0.61	3.25	1.25	0.235	0.70	63%
4 - Terminal T1	1991	"Non-Ess."	ORCSW (frame)	3	0.3	1	2.75		8		0.103	ASD	0.61	5	1.25	0.153	0.70	97%
4 - Terminal T2	1994	Std or Sp	ORCSW (bearing)	3	0.3	1	2.75		6		0.138	ASD	0.61	4	1.25	0.191	0.70	103%
4 - Terminal T3	1997	Std or Sp	ORCSW (frame)	3	0.3	1	0.36		5.5		0.164	USD	0.61	5	1.25	0.153	1.00	107%
4 - Terminal T3	1997	Std or Sp	OSCBF	3	0.3	1	0.36		5.5		0.164	USD	0.61	3.25	1.25	0.235	1.00	70%
4 - Terminal T4	1997	Std or Sp	OSCBF	3	0.3	1	0.36		5.5		0.164	USD	0.61	3.25	1.25	0.235	1.00	70%
4 - Terminal T4	1997	Std or Sp	ORCSW (frame)	3	0.3	1	0.36		5.5		0.164	USD	0.61	5	1.25	0.153	1.00	107%
5 - ARFF	1991	EF	IRMSW (bearing)	3	0.3	1.25	2.75		6		0.172	ASD	0.61	4	1.5	0.229	0.70	107%
6a - HQ/P2(N) E-W	2006 IBC	Std	SSMRF and SRCSW		0.61	1			7		0.087	USD	0.61	7	1	0.087	1.00	100%
6a - HQ/P2(N) N-S	2007 IBC	Std	SSMRF		0.61	1			8		0.076	USD	0.61	8	1	0.076	1.00	100%
6b - P2(S) E-W	2006 IBC	Std	SSMRF and SRCSW		0.61	1			7		0.087	USD	0.61	7	1	0.087	1.00	100%
6b - P2(S) N-S	2007 IBC	Std	SSMRF		0.61	1			8		0.076	USD	0.61	8	1	0.076	1.00	100%
13 - Gnd.Maint. Admin. & Shops Facility BLDG B	1979 UBC	Std	Precast panels	2	0.375	1		0.14		1	0.053	ASD	0.61	2.5	1	0.242	0.70	31%
14 - Gnd. Maint. Facility BLDG A	1979 UBC	Std	Precast panels	2	0.375	1		0.14		1	0.053	ASD	0.61	3	1	0.202	0.70	37%
15 - Gnd. Maint. Facility BLDG C	1979 UBC	Std	Precast panels/ORMSW	2	0.375	1		0.14		1	0.053	ASD	0.61	2.5	1	0.242	0.70	31%
16 - T-6 Maintenance BLDG (E-W)	1970 UBC	Std	OSMRF	2	0.5		0.1			0.67	0.034	ASD	0.71	3.5	1	0.204	0.70	23%
16 - T-6 Maintenance BLDG (N-S)	1970 UBC	Std	Ordinary tension-only rod braced frames/precast concrete walls	2	0.5		0.1			1	0.050	ASD	0.71	3	1	0.238	0.70	30%
17 - T-6 Electrical Shop (E-W)	1985 UBC	Std	OSMRF	2	0.375	1		0.14		0.67	0.035	ASD	0.71	3.5	1	0.204	0.70	25%
17 - T-6 Electrical Shop (N-S)	1985 UBC	Std	Ordinary tension-only rod braced frames	2	0.375	1		0.14		1	0.053	ASD	0.71	3.25	1	0.220	0.70	34%

* C varies 2.55 to 2.75
IRMSW = Intermediate Reinforced Masonry Shear Wall
ORCSW = Ordinary Reinforced Concrete Shear Wall
ORMSW = Ordinary Reinforced Masonry Shear Wall
OSCBF = Ordinary Steel Concentric Braced Frame
OSMRF = Ordinary Steel Moment-resisting Frame
SMRF = Special Moment-resisting Frame
SRCSW = Special Reinforced Concrete Shear Wall
SSMRF - Special Steel Moment-resisting Frame

Building Foundations Summary

Asset ID	Asset Name	Foundations (pile reference and depth, or fdn type)	Pile Type	Design Capacity
1	CUP	1972: -52' to -65'	Timber	-25T
		1992: 5' IDS	Augercast	-35T
		1994: -50' (W. Exp/Emerg. Gens)	Augercast	-40T
		1999: 5' IDS	Augercast	-40T/+25T
3 C1	Terminal Concourse C (C1)	TES2: -70', ~25' IDS	Pipe (closed end)	-65T/+50T
		TES2: -80', ~35' IDS	Pipe (open end)	-65T/+50T
3 C2	Terminal Concourse C (C2)	TES2: -70', ~25' IDS	Pipe (closed end)	-65T/+50T
		TES2: -80', ~35' IDS	Pipe (open end)	-65T/+50T
3 C3	Terminal Concourse C (C3)	TES2: -70', ~25' IDS	Pipe (closed end)	-65T/+50T
		TES2: -80', ~35' IDS	Pipe (open end)	-65T/+50T
4 T1	Main Psgr Terminal Bldg (T1)	1956: 5' IDS	Timber/concrete composite	-25T/?
		1973: -50'	Pipe	-25T/+25T
		TAP micropiles: 25'+ IDS(?)	Micropile	-125T/+80T
4 T2	Main Psgr Terminal Bldg (T2)	TES2: -70', ~25' IDS	Pipe (closed end)	-65T/+50T
		TES2: -80', ~35' IDS	Pipe (open end)	-65T/+50T
		TES2 micropiles: ~25'+ IDS	Micropile	-125T/+100T
		Conc B: 15' IDS	Pipe (closed end)	-65T/+35T
		1973 (Rdwy): -50'	Pipe	-25T/+25T
4 T3	Main Psgr Terminal Bldg (T3)	1956: 5' IDS	Timber/concrete composite	-25T/?
		1986: 15' IDS	Pipe (closed end)	-65T/+35T
		TES3: -80'	Pipe (open end)	-65T/+50T
		TES3 micropiles: 25'+ IDS (?)	Micropile	-100T/+100T
4 T4	Main Psgr Terminal Bldg (T4)	See 1956 and micropiles above	See above	Varies
5	ARFF	Mat: 12" s.o.g. thickened to 18"-24" at walls/columns	-	750 psf
6a	Admin (HQ) Offices+Parking (P2)	40' IDS	Pipe (open end)	-105T/+75T
6b	Parking (P2) Structure(South)	40' IDS	Pipe (open end)	-105T/+75T
13	Ground Maint. Admin & Shops	Spread	-	2000 psf
14	Ground Maint. Facility	Spread	-	2000 psf
15	Ground Maint. Facility	Spread	-	2000 psf
16	CDC Whse - Maintenance	Spread	-	-
17	Electrical Shop Bldg	Spread	-	-

Notes: All dimensions are maximum tip elevation, uno
Dimensions with "IDS" are embedment lengths into the medium dense to dense sand layer
All information is from existing drawings and/or soils reports, uno
Pile capacities: "-" = compression, "+" = tension
ROM DL ~50% of design load (will vary with construction type and number of levels, etc)
Subject to liquefaction settlement

Building Code Summary

The first terminal structure at PDX was built almost 60 years ago. The building codes and knowledge base related to earthquakes in this region, behaviors of soils, and the performance of different lateral force resisting systems, has grown immensely since that time. Following is a description of the progression of the building codes over that time in how they have addressed seismic design. Additional information on the Geotechnical aspects of earthquakes and soil behaviors can be found in those portions of this study.

Building Performance

Building codes have historically been primarily concerned with safeguarding against major failures and loss of life; not to limit damage, maintain function or business continuity, or to provide for economical repair. Performance objectives are categories with defined minimum desired behavior or condition of a structure when subjected to a given seismic event. The Code prescribes minimum performance objectives; however, any performance objective can be defined by an Owner, provided it meets or exceeds the Code minimum. The seismic event is usually expressed in average return period (smaller, more frequent events = shorter return period, larger, less common events = longer return period), or probability of exceedance in a given timeframe (e.g. 10% chance of exceedance in 50 years = 475 year return period, 2% chance of exceedance in 50 years = 2475 return period) . The current Oregon Structural Specialty Code (OSSC) design methodology follows. Refer to the section following for a description of the performance objectives.

Current Code Design Methodology

Collapse Prevention: Implicitly based on the 2475-year event, also referred to as the maximum considered earthquake (MCE).

Life Safety: OSSC Code design is based on 2/3 of the MCE-level earthquake. This factor is based on studies that have indicated that a building at the Life Safety level typically has at least 50% reserve capacity before the onset of collapse. This Code level design is to provide a minimum standard to maintain the building in a condition with minimal falling hazards and adequate post-earthquake stability for people to safely exit. It does *not* take into account economic considerations of the Owner's business, or other secondary effects. The return period that this correlates to varies from region to region, with their differing earthquake types and characteristics, as well as the local site soil. In parts of California, this design level roughly correlates to a 500-year earthquake, which corresponds to the desired earthquake design level of older codes.

Immediate Occupancy: The OSSC Code does not explicitly address this performance objective. Since 1976, the Code has included Importance factors for Essential, Hazardous and Special (primarily higher occupancy) structures. An Essential Facility is defined in the Code as "buildings or other structures that are intended to remain operational in the event of … earthquakes." Currently, the Code prescribes increasing the force level by 50% for Essential or Hazardous Facilities, and

25% for Special occupancies. It also prescribes more stringent deflection criteria, to mitigate effects on cladding, window systems, utilities, etc., and equipment that is to be operational after an event.

It should be noted that the Code factors above are independent of the lateral force resisting system used. The recently released ASCE 41-13 approaches the design differently. Rather than constant factors separating the performance objectives, it provides unique factors for different elements of the system and for each performance objective. This method accounts for the different characteristics of a steel moment frame, a steel diagonal brace, or a concrete shear wall, much more accurately than the traditional method, and is expected to be the trend in future codes.

Performance Objectives

Following are the descriptions of the performance objectives addressed by Code, as found in ASCE 41. It should be noted that building performance is a continuum, not discreet levels, and may be affected by factors such as over- or under-design of certain elements, contractor workmanship, redundancies or lack thereof. A building may perform at, or above, the objective for certain elements, and below for certain others; but could be considered to meet the objective if the primary goal(s) of the objective is achieved.

Operational

A building performance level defined as Immediate Occupancy Structural Performance Level combined with Operational Nonstructural Performance Level. Structural damage is as defined in Immediate Occupancy below; with essential utilities being either serviceable, or on emergency sources. Continued occupancy and use of the building is possible, although possibly in a slightly impaired mode.

Typical concrete wall: Minor cracking.
Typical steel brace: Minor yielding or buckling.
Typical steel moment frame: Minor local yielding and buckling at a few locations.
Typical glazing: No cracked or broken panes.
Typical partitions: Minor cracking. No impact of functionality.
Typical utilities: Negligible damage. Emergency systems are functional, possible from emergency sources.

Immediate Occupancy

Building remains safe to occupy and essentially retains its pre-earthquake strength and stiffness. Structural damage as well as non-structural damage is low; however minor repairs may be required. Functionality is limited due to impact to non-structural items and utilities serving the building.

Typical concrete wall: Minor cracking.
Typical steel brace: Minor yielding or buckling.
Typical steel moment frame: Minor local yielding and buckling at a few locations.
Typical glazing: Some cracked panes, none broken. Some loss of weather-tightness
Typical partitions: Some cracking, especially at openings.

Typical utilities: Minor damage and leaking. MEP secure in place, but not potentially not operable. Emergency systems functional.

Enhanced Life Safety

Performance range between Life Safety and Immediate Occupancy. Occupancy/Risk Category III (e.g. PDX terminal and concourses) structures would be expected to fall in this range.

Life Safety

Building has damaged components but retains a margin of safety against the onset of partial or total collapse, with some residual strength and stiffness remaining in all stories. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside of the building. Overall risk of life-threatening injuries as a result of structural damage is low. Non-structural items and utilities may be damaged. It should be possible to repair the structure; however it may not be economical. Repairs may be required before reoccupancy.

Typical concrete wall: Some cracking and spalling. Damage around openings. Extensive cracks and some crushing at coupling beams between walls.

Typical steel brace: Many braces yield or buckle, but do not completely fail. Many connections might fail.

Typical steel moment frame: Local buckling of beams and severe joint distortion. Some isolated flange connection fractures.

Typical glazing: Extensively cracked glass with potential loss of weather-tightness. Overhead panes do not shatter or fail.

Typical partitions: Some severe cracking and racking.

Typical utilities: Some ducts/pipes broken and some supports failing. Some pipes and ducts falling. Emergency systems may not function.

Collapse Prevention

Building has damaged components with large permanent drifts and continues to support gravity loads, but retains no margin of safety against partial or total collapse. Significant risk of injury caused by falling hazards from structural debris might exist. Building is not safe for reoccupancy as aftershocks could cause collapse, and might not be technically practical to repair. Statistically, 10% of buildings are expected to partially or totally collapse if subjected to the design Collapse Prevention earthquake.

Typical concrete wall: Major cracks along with extensive crushing and buckling of reinforcement. Coupling beams between walls virtually disintegrated.

Typical steel brace: Extensive yielding and buckling of braces. Many braces and connections might fail.

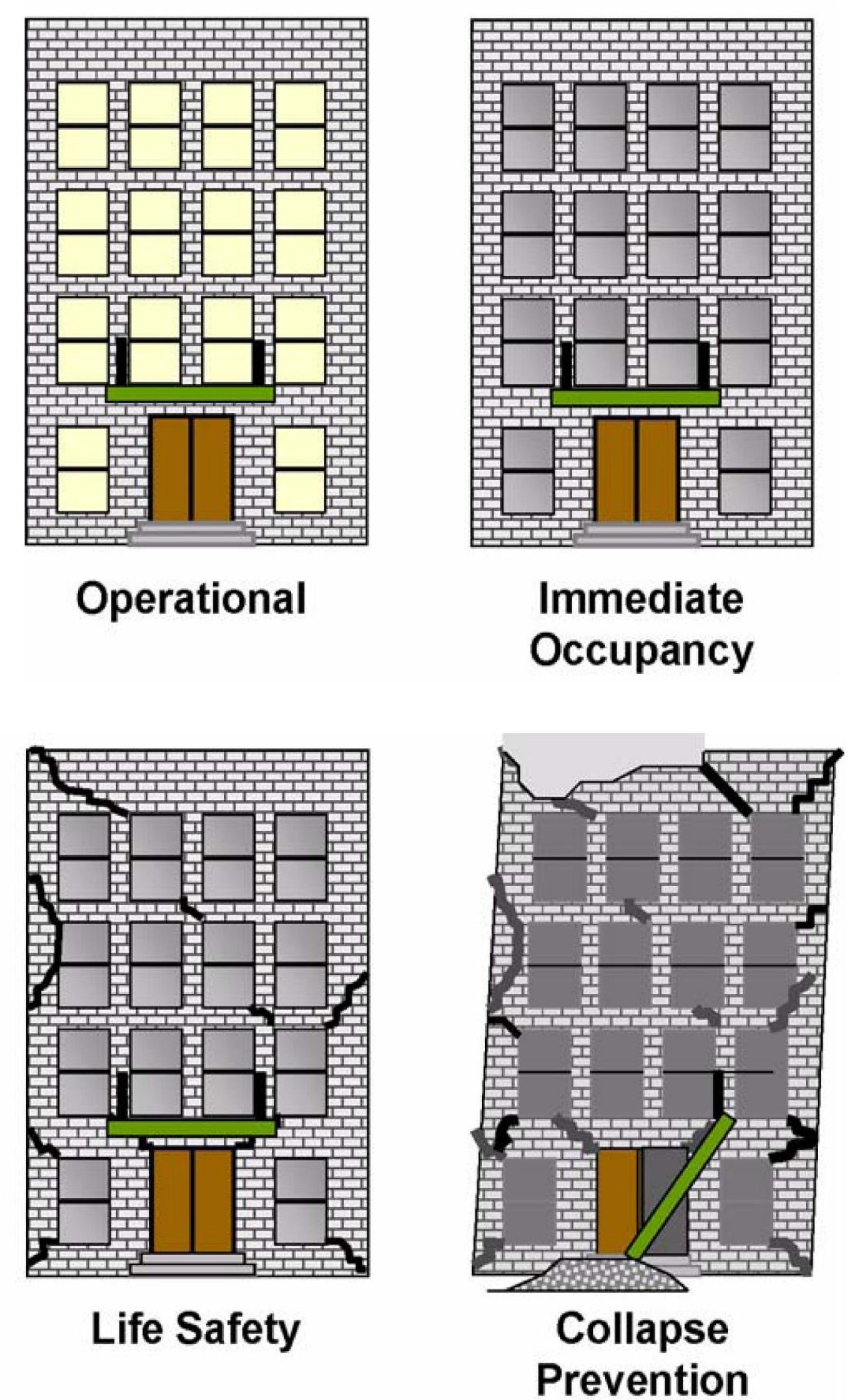
Typical steel moment frame: Extensive distortion. Many fractures at flange connections.

Typical glazing: Condition not considered.

Typical partitions: Condition not considered.

Typical utilities: Condition not considered.

The following figure graphically shows the performance levels, from “lights on” to near collapse.



Earthquake Hazard Level

Prior to the introduction of the Uniform Building Code in Oregon, seismic design for buildings was very minimal. After the UBC became the standard code for Oregon in the early 1970's, until the early 1990's, Oregon was considered a moderate earthquake region, with few significant earthquakes in recent history: but with substantially greater seismic design than before. When research uncovered the regular occurrence of catastrophic subduction zone earthquakes, the design forces increased substantially again. This near doubling of design force occurred between the construction of the west end of Concourse D (originally Concourse K) and the Terminal Expansion North (TEN) projects.

Ductility

As seismic design has progressed, the importance of detailing of the system components in order to achieve the desired performance level has become more apparent and increasing addressed and refined in the Code. This has also resulted in certain systems that lack adequate ductility (e.g. tension-only rod bracing, or connections that fail before the main member) being penalized by reduced ductility factors. The result is in better overall building performance, even for design to the same, or in some cases somewhat lower, force level.

Code Progression

Following is a brief summary of the progression of the seismic design aspects of the commercial building code in Oregon. The chart following graphically shows the progression of seismic design forces along with some of the key assets and major Airport projects.

First City of Portland Code Adoption – 1970 UBC

The City of Portland adopted the 1970 Uniform Building Code (UBC) in 1972. Prior to this, the City used the Building Ordinances of the City of Portland. Portland was designated as Seismic Risk Zone 2 in this Code, equating (as indicated in the Code) to the structure being subjected to a moderate damage seismic event equivalent to intensity VII on the Modified Mercalli Intensity Scale (M .M.). See the end of this section for a description of the Modified Mercalli Intensity Scale.

First Statewide Building Code – 1973 UBC

Oregon adopted the 1973 Uniform Building Code (UBC) as the first state building code in 1974, to bring all city and county jurisdictions under one design basis building code. The code was known as the State of Oregon Structural Specialty Code and Fire and Life Safety Code (OSSC). The seismic provisions were similar to the 1970 UBC, but with some additional criteria for improved ductility of the seismic force resisting system. All of Oregon was designated as Seismic Risk Zone 2.

1976 UBC (1978 OSSC)

The seismic provisions underwent minor modifications in the 1976 Code. A new Seismic Risk Zone 4 was added and the factors to determine the base shear coefficient were adjusted slightly; reducing the Zone factor and adding a new coefficient for site-structure resonance. The Code also added an Importance

Factor: 1.5 for Essential Facilities, and 1.25 for high-occupancy buildings. The net result of these changes for a Standard Occupancy building in Oregon was an approximately 5 percent increase in base shear.

1979, 1982, 1985 UBC (1980, 1983, 1986 OSSC)

The seismic provisions of the 1979 through 1985 Uniform Building Codes (1980 through 1986 OSSC) changed very little during these years.

1988 UBC (1990 OSSC)

The seismic design factors in the 1988 UBC changed significantly from the previous version. This Code introduced the "R" factor, shifting this coefficient related to the ductility of the system to the denominator, along with defining most of Oregon as a new Seismic Zone 2B (a small area along the southern border was Seismic Zone 3). Importance Factors for Essential Facilities and Special Occupancies were revised, as well as adding this factor for Hazardous Facilities. The net result of these changes for Oregon was an approximately 30 percent increase in base shear.

1991 UBC (1993 OSSC)

As a result of recent geotechnical research on subduction zone earthquakes, the State of Oregon designated the counties west of the Cascade Range of mountains as Seismic Zone 3, while the eastern counties remained at Seismic Zone 2B. This change results in a 50 percent increase in base shear from the previous Seismic Zone 2B design criteria. The Terminal Expansion North (TEN) project was designed for this new Seismic Zone designation.

1994 UBC (1996 OSSC)

The changes in this Code were minor, and included some clarifications to Occupancy Categories. The design base shear did not change for most buildings under this Code.

1997 UBC (1998 OSSC)

This Code made several significant revisions. The seismic zones for Oregon remained the same except for a portion along the southern Oregon coast, which was upgraded to Seismic Zone 4. The changes to the base shear design, included new factors that more explicitly considered the site soil charateristics depending on whether the building was acceleration sensitive (short period) or velocity sensitive (long period). It also adjusted the "R" factor values and increased the Importance Factor for Essential and Hazardous Facilities from 1.25 to 1.5. For Seismic Zone 4 structures, there was also an addition of near-source factors. The base shear for a typical concrete shear wall in Seismic Zone 3 experienced an increase of approximately 4 percent. This Code remained in effect for approximately six years.

With this Code adoption, there were also many more stringent ductility requirements for steel moment frames and braced frames, resulting in large part from research into performance of systems observed after the 1994 Northridge earthquake.

In the 1990's, the three regional building codes organizations, the Building Officials Code Administrators International (BOCA), the Southern Building Code Congress International (SBCCI), and the International Conference of Building Officials (ICBO, publishers of the UBC), combined their efforts and formed the International Code Council (ICC). The ICC developed and published a new model code, the International Building Code (IBC), to eliminate regional discrepancies.

2003 IBC (2004 OSSC)

In 2004, the State of Oregon made the move from the Uniform Building Code to the International Building Code (IBC) and adopted the 2003 International Building Code as amended to be the 2004 Oregon Structural Specialty Code. The IBC upgraded its design parameters by requiring the design to a 2,500-year return period earthquake versus a 500-year return period of an earthquake in the previous edition of the UBC codes. This change incorporated a substantial shift in earthquake regulations and how the seismic base shear was determined. Site seismic factors at short (0.2 seconds) and long (one second) periods, which were determined by using the specific latitude and longitude in conjunction with the United States Geological Survey (USGS) mapping, were combined with specific site soil characteristics. This then allowed the creation of a response spectrum specific to the site for use in design. The types of structural systems were expanded considerably and, when used with the revised base-shear formulation, gave very site-specific seismic loading. The net result of the new technology and more precise method of loading determination and generally a moderate lowering of the seismic base-shear forces, varying depending on the system, location, and site characteristics. An Importance Factor of 1.25 was also added back in the Code for Special Occupancy structures.

2006 IBC (2007 OSSC)

In 2007, the 2006 International Building Code was adopted, with modifications, as the 2007 Oregon Structural Specialty Code (OSSC). The most significant change in this code was the referencing of the major code groups as part of the OSSC in an attempt to reduce repetition and inconsistency. ASCE 7-05 was referenced as the code for developing seismic forces for building design. This was not a big shift because ASCE 7-05, in conjunction with NEHRP seismic provisions, was already the basis for design in the previous code. No major change in seismic base-shear design values resulted from the adoption of the 2007 OSSC.

2009 IBC (2010 OSSC)

Keeping in line with the previous code adoption processes, the 2009 IBC was adopted as the 2010 Oregon Structural Specialty Code. The basic code, as it relates to earthquake design, carried forward from the previous code cycle with very little change.

2012 IBC (2014 OSSC)

The 2012 IBC adopted the newer ASCE 7-10 for seismic design. Changes involved updated risk-targeted seismic hazard maps, and revising Occupancy Categories to Risk Categories. Overall, seismic revisions in the new Code will typically be minor for building structures.

Importance Factor

The concept of an Importance Factor to increase the seismic resilience of certain structures that include emergency services or shelter or higher occupancy loads, was introduced in the 1976 UBC. An importance factors for facilities dealing with hazardous materials was added in 1988. These factors, and to some degree, the category definitions, has changed during the years. The page titled History of Code Seismic Importance Factors shows the progression over the years of this factor. The latest Code, the 2014 OSSC, revised Occupancy Category to Risk Category, with essentially the same category definitions. The page titled Occupancy/Risk Category shows an abbreviated definition of the categories according to the current Code.

Modified Mercalli Intensity Scale with the corresponding Richter Scale Magnitude (RM)

I. Not felt except by a very few under especially favorable conditions. RM = 3.5

II. Felt only by a few people at rest, especially on upper floors of buildings. RM = 4.2

III. Felt quite noticeably by people indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations are similar to the passing of a truck. Duration estimated. RM = 4.3

IV. Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, and doors disturbed; walls make cracking sound. Sensation like a heavy truck striking a building. Standing motor cars rocked noticeably. RM = 4.8

V. Felt by nearly everyone; many awakened. Some dishes and windows broken. Unstable objects overturned. Pendulum clocks may stop. RM = 4.9-5.4

VI. Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight. RM = 5.5-6.0

VII. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken. RM = 6.1

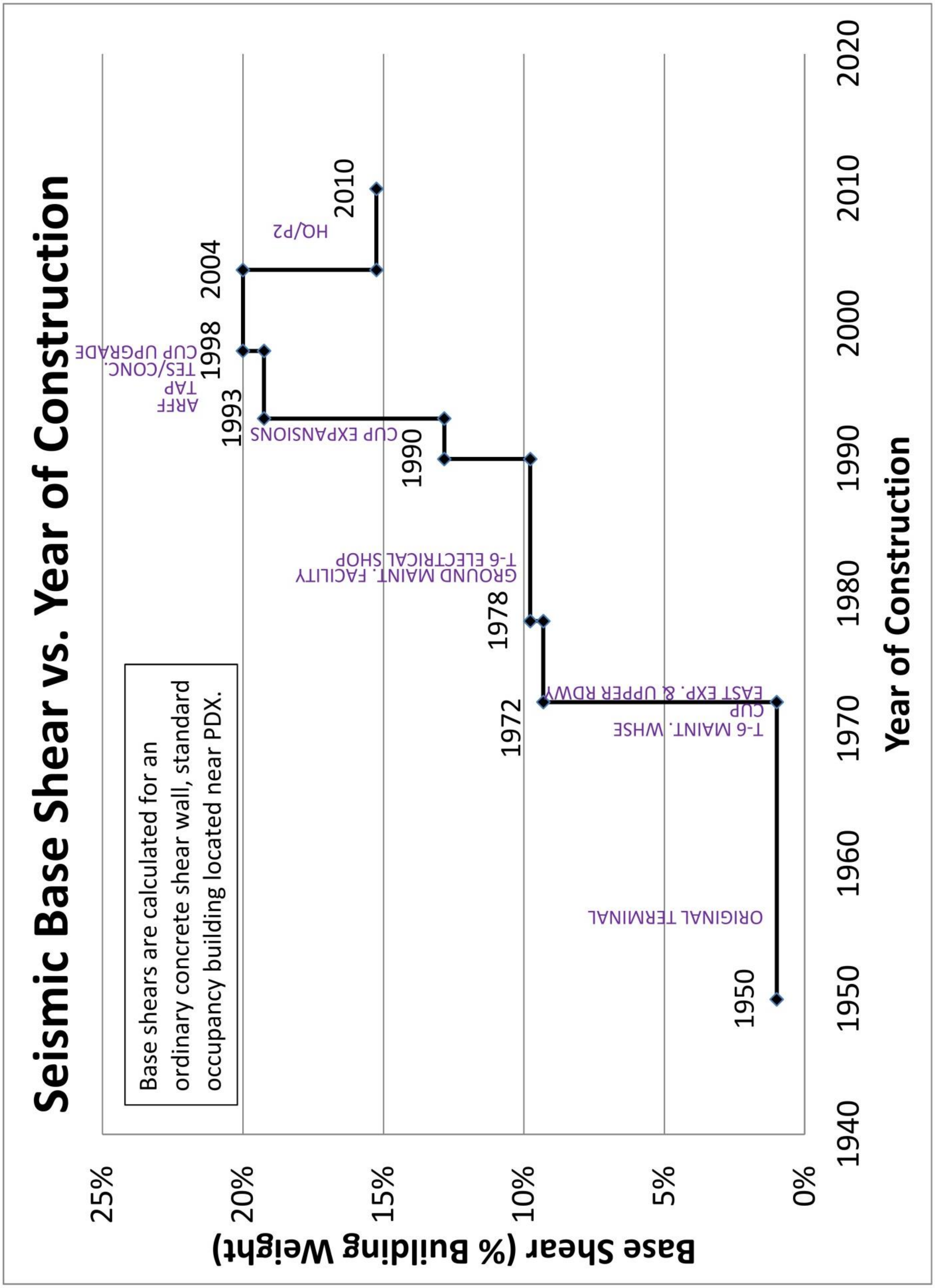
VIII. Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, and walls. Heavy furniture overturned. RM = 6.2

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. RM = 6.9

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent. RM = 7.0-7.3

XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly. RM = 7.4-8.1

XII. Damage total. Lines of sight and level are distorted. Objects thrown into the air. RM = >8.1


Seismic Base Shear vs. Year of Construction
Base shears are calculated for an ordinary concrete shear wall, standard occupancy building located near PDX.
Base Shear (% Building Weight)
0%
5%
10%
15%
20%
25%
Year of Construction
1940
1950
1960
1970
1980
1990
2000
2010
2020
1950
ORIGINAL TERMINAL
1972
T-6 MAINT. WHSE
CUP
EAST EXP. & UPPER RDWY
1978
GROUND MAINT. FACILITY
T-6 ELECTRICAL SHOP
1990
CUP EXPANSIONS
1993
ARFF
TAP
TES/CONC.
CUP UPGRADE
1998
2004
HQ/P2
2010

kpff

History of Code Seismic Importance Factors

	1974 OSSC		1978 OSSC		1980 OSSC		1983 OSSC	
	1973 UBC		1976 UBC		1979 UBC		1982 UBC	
	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.
EF (Essential)	-	-	1.5	-	1.5	-	1.5	-
HF (Hazardous)	-	-	-	-	-	-	-	-
Sp (Special)	-	-	1.25	-	1.25	-	1.25	-
Std (Standard)	-	-	1	-	1	-	1	-
Low	-	-	-	-	-	-	-	-
Design Method	ASD		ASD		ASD		ASD	

	1986 OSSC		1990 OSSC		1993 OSSC		1996 OSSC	
	1985 UBC		1988 UBC		1991 UBC		1994 UBC	
	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.
EF (Essential)	1.5	-	1.25	I	1.25	I	1.25	1
HF (Hazardous)	-	-	1.25	II	1.25	II	1.25	2
Sp (Special)	1.25	-	1	III	1	III	1	3
Std (Standard)	1	-	1	IV	1	IV	1	4
Low	-	-	-	-	-	-	1	5
Design Method	ASD		ASD		ASD		ASD	

	1998 OSSC		Not Adopted		2004 OSSC		2007 OSSC	
	1997 UBC		2000 IBC		2003 IBC		2006 IBC	
	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.	I_e	Occ. Cat.
EF (Essential)	1.5	1	1.5	III	1.5	IV	1.5	IV
HF (Hazardous)	1.5	2	1.5	III	1.5	IV	1.5	IV
Sp (Special)	1	3	1.25	II	1.25	III	1.25	III
Std (Standard)	1	4	1	I	1	II	1	II
Low	1	5	1	IV	1	I	1	I
Design Method	LRFD		LRFD		LRFD		LRFD	

	2010 OSSC		2014 OSSC	
	2009 IBC		2012 IBC	
	I_e	Occ. Cat.	I_e	Risk Cat.
EF (Essential)	1.5	IV	1.5	IV
HF (Hazardous)	1.5	IV	1.5	IV
Sp (Special)	1.25	III	1.25	III
Std (Standard)	1	II	1	II
Low	1	I	1	I
Design Method	LRFD		LRFD	

Red = Revision to preceding Code

Design Method: ASD = Allowable Stress Design

LRFD = Load and Resistance Factor Design

Occupancy/Risk Category

I Buildings representing a low hazard to human life:

- Agricultural facilities
- Minor storage

II All buildings except noted otherwise

III Buildings representing a substantial hazard to human life or substantial economic impact or containing hazardous materials:

- >300 people congregate in one area
- Daycare >150
- Elementary or secondary schools >250
- Colleges >500
- Power stations
- Water and sewage treatment plants
- Telecommunications

IV Essential facilities:

- Hospitals
- Fire, rescue, police stations
- Designated emergency shelters
- Aviation control towers
- Highly hazardous materials

Appendix 4 – Marine Facilities Seismic Vulnerability Assessment

(BergerABAM)

BergerABAM

PORT OF PORTLAND

Marine Facilities Seismic Vulnerability Assessment

Submitted to

HNTB Corporation
Bellevue, Washington

A14.0235.00

24 February 2015

Seismic Risk Assessment

Port of Portland
Marine Facilities Seismic Risk Assessment

Submitted to

HNTB Corporation
Bellevue, Washington

24 February 2015

REGISTERED PROFESSIONAL
ENGINEER
71234PE
OREGON
July 9, 2002
THOMAS SCOTT McMAHON
3.3.2015
RENEWS: 12/31/16

Submitted by

BergerABAM
700 Northeast Multnomah Street, Suite 900
Portland, Oregon 97232-4189

A14.0235.00

SEISMIC RISK ASSESSMENT

Port of Portland
Marine Facilities Seismic Risk Assessment

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EXECUTIVE SUMMARY

The Port of Portland is undertaking a seismic risk assessment of the Port's key assets. As a first phase of this effort, the Port identified a prioritized list of assets for evaluation that includes buildings, runways, and marine structures. The purpose of the project is to provide a high-level evaluation of seismic performance and vulnerability of each key asset and provide cost/benefit ratios to identify and prioritize future seismic mitigation projects. This report summarizes the evaluation of the marine facilities and provides damage and downtime estimates to be used in the benefit/cost analysis to be completed by ImageCat, Inc. Comprehensive seismic hazard, structural, damage, and loss analyses were not performed.

The marine facilities included in this seismic risk assessment include:

Terminal 4

- Berths 410 and 411 – Soda Ash Export Facility

Terminal 5

- Berth 501 – Grain Export Facility
- Berth 503 – Potash Export Facility

Terminal 6

- Berth 601 – Automobile Facility
- Berths 604 and 605 – Container Terminal

The assessment of the prioritized assets indicates the waterfront structures at Berths 410, 411 and 503 will likely perform adequately at an earthquake with a 72-year return period. Berths 501 and 601 could potentially experience significant damage at this same earthquake level. At an earthquake event with a 475-year return period and above, all waterfront structures evaluated are vulnerable to the effects of large estimated soil lateral spreading displacements and may experience significant damage, potentially requiring reconstruction to reestablish service. The effects on the facilities at earthquake events were assessed at three return periods; 72, 475, and 975 years. Intermediate return periods were not assessed. A more comprehensive seismic hazard, structural, damage, and loss analysis is recommended for a subsequent phase of the seismic risk assessment program to further evaluate risk and potential mitigation measures.

AUTHORIZATION

On 4 September 2014, BergerABAM was authorized by HNTB to provide engineering services for seismic risk assessments of selected marine facilities at the Port of Portland. The project is part of a Seismic Risk Assessment task order (HNTB Project Number 58824-DS-002) that HNTB is completing for the Port.

INTRODUCTION

The Port of Portland (Port) is undertaking a seismic risk assessment of the Port's key assets. As a first phase of this effort, the Port identified a prioritized list of assets for evaluation that includes buildings, runways, and marine structures. The purpose of the project is to provide a high-level evaluation of seismic performance and vulnerability of each key asset and provide cost/benefit ratios to identify and prioritize future seismic mitigation projects. Comprehensive seismic hazard, structural, damage, and loss analyses were not performed, but are recommended to be completed in a subsequent phase of the Port's overall seismic risk assessment. This report summarizes the evaluation of the marine facilities.

BergerABAM, as a subconsultant to HNTB, reviewed seismic hazards, assessed seismic vulnerability, estimated possible consequences including damage costs and facility downtime, and developed potential mitigation strategies and costs for a number of the Port's marine facilities. Once this was completed, benefit/cost evaluations were developed by ImageCat, Inc. of Long Beach, California.

The marine facilities included in this seismic risk assessment include:

Terminal 4

- Berths 410 and 411 – Soda Ash Export Facility

Terminal 5

- Berth 501 – Grain Export Facility
- Berth 503 – Potash Export Facility

Terminal 6

- Berth 601 – Automobile Facility
- Berths 604 and 605 – Container Terminal

BACKGROUND AND METHODOLOGY

On a system-wide basis, a seismic risk assessment of the Port's infrastructure is part of an overall seismic risk management plan. The seismic assessments of the Port's buildings, runways, and marine structures are intended to provide insight into the Port's financial risk from anticipated seismic events and to help the Port establish a level of acceptable risk – the point at which additional cost to reduce the risk is decided to be excessive and unnecessary (Werner, et al., 2008). We understand that this initial phase of work will help the Port to prioritize its upgrade and replacement projects and to help guide future development with a better understanding of the risk from seismic events. Five marine facilities are included in this assessment, comprising seven berths.

The assessment began with the identification of seismic hazards present at each site. The primary seismic hazards for the Port's marine structures include ground shaking, liquefaction, lateral spreading, and settlement. Secondary seismic hazards include fire and loss of containment of hazardous materials. Only the primary seismic hazards are included in this assessment. GRI and New Albion Geotechnical, Inc. (NA) characterized the general subsurface conditions, developed ground motion recommendations (in the form of acceleration response spectra curves) to be used for analysis, and estimated permanent vertical and horizontal ground deformation at each site. Acceleration response spectra curves were developed for five of hazard levels. The ground motions were developed specifically for this assessment, are not intended for design, and do not match code-level ground motions. Geotechnical methods, findings, and recommendations by GRI and NA can be found in a separate report.

For the marine structures, the hazard levels of interest were developed following performance-based design concepts that have commonly been used in the design of port marine structures on the West Coast. Different performance objectives were targeted for each seismic load level, allowing for life-safety design at the highest seismic level, but also allowing for targeted performance criteria at lower seismic levels. Using ASCE *Seismic Design of Piers and Wharves* code as guidance for this assessment, a three-level seismic hazard level and performance level was used. For new designs corresponding to a "High" design classification (essential structures to the regional economy or post-event recovery), the three hazard levels (including description of the ground motion probability of exceedance (PE) and the performance level) are:

- Operating Level Earthquake (OLE): 72-year return period (50 percent PE in 50 years); minimal damage with near-elastic structural response with little or no residual deformation, little or no loss of serviceability of the structure, and no loss of containment of materials.
- Contingency Level Earthquake (CLE): 475-year return period (10 percent PE in 50 years); controlled and repairable damage with response in a ductile manner, limited inelastic deformations with repairable damage, loss of serviceability for no more than several months, and no loss of containment of materials.
- Design Earthquake (DE): Life safety protection at the design earthquake per ASCE 7-05. For design, the DE is obtained by taking two-thirds of the spectral response acceleration for the Maximum Considered Earthquake (MCE), which is a probabilistic earthquake with a 2,475-year return period (2 percent PE in 50 years). For the purposes of this evaluation, a 975-year return period (5 percent PE in 50 years) was used as the upper bound earthquake based on a comparison of the Peak Ground Acceleration (PGA) for a site class at the B/C boundary; however, this will differ once site class effects are considered.

The next step was to assess the vulnerability of each structure to the selected seismic hazards. BergerABAM performed a general review of the original structural drawings for each facility to understand the seismic force resisting system and to evaluate potential structural deficiencies that could lead to structural damage during a seismic event. In addition to the drawing review, original and retrofit seismic calculations, models, and reports for the waterfront structures were reviewed when available from the Port's record library.

In 2013, BergerABAM conducted site visits to each of the identified facilities as part of an overall marine facilities condition assessment. On 23 April 2014, a site visit to Berths 410, 411, 503, and 601 was performed by representatives from both BergerABAM and GRI to verify that no significant changes had occurred since the previous assessment in 2013. The general conditions of the berth structures were noted.

A screening-level vulnerability assessment was made at each of the three hazard levels based on the review of available data and considering the current condition of each structure to estimate expected damage and repair downtime and costs. Where expected damage is significant and not repairable, downtime and costs for demolition and reconstruction is presented. High-level qualitative assessments were made using engineering judgment, supplemented with streamlined quantitative assessments using simple structural models where appropriate. Assessments were not made to establish expected damage and downtime at intermediate hazard levels.

A more comprehensive evaluation (not included in the scope of this report) may include the development of fragility models that capture the probability of occurrence of a structure damage state as a function of the seismic hazard. Vulnerability and fragility functions will need to be developed in subsequent phases of the project for evaluating specific proposed mitigation measures.

Mitigation measures to improve the seismic performance of the existing marine structures to meet the CLE are presented in this report. Several alternate mitigation measures may be appropriate at a given facility; however, generally, a single mitigation measure was evaluated for the purpose of this assessment. This information was used in a benefit-cost analysis to evaluate potential seismic improvements. In cases where seismic upgrades to the existing structure are not feasible or reasonable, mitigation alternatives may include reconstruction.

Estimated downtime and repair/reconstruction and mitigation costs assume a nominal time and cost for planning and permitting. Planning and permitting downtime may vary significantly, anywhere from several months to five years or more. These estimates do not consider scarcity of resources expected after a major seismic event.

A summary of the findings, including estimates of repair costs and downtime is provided in Appendix A.

The seismic risk assessment contained in this report is limited to the berth structure and does not account for the port and tenant-owned mechanical equipment (cranes, conveyors, etc.) that may be supported by the structure. A future, broader system assessment may also include other elements, the failure of which would jeopardize the use of the facility. These elements include utilities, slopes, transportation systems, etc.

MARINE FACILITIES SEISMIC RISK ASSESSMENT

Terminal 4, Berth 410

Structure Description and Condition

Berth 410 is part of the Kinder Morgan dry bulk facility. Berth 410 is a timber pile-supported pier structure constructed in 1962 and was built as an extension of the Berth 411 wharf structure. No seismic isolation joint between the two berth structures was provided; however, we did not see evidence that the two structures were intended to work together to resist seismic loads. Berth 410 is approximately 490 feet long by 60 feet wide. In the past, the primary use of the pier was to provide tail track for shuttling rail cars through the dumper building and to provide mooring points for ships using Berths 410 and 411; however, rails were removed from the deck in recent years. The pier deck elevation is approximately +32.7 feet and the design mudline elevation at the face of pier is -35 feet.

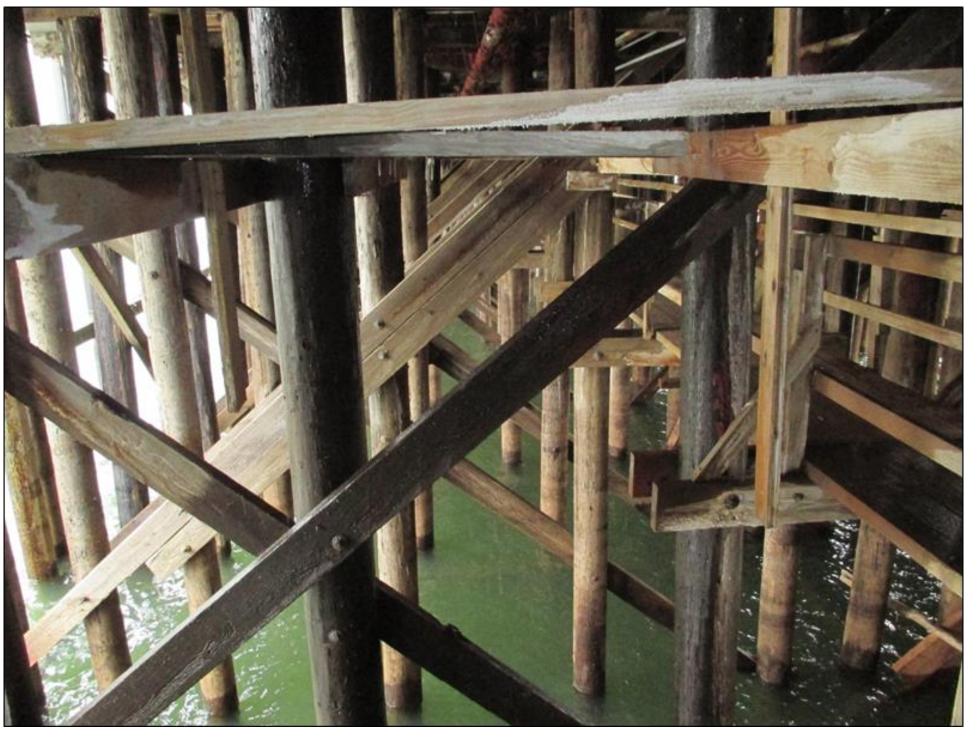

Photo 1 – Berth 410 typical timber piles and bracing configuration

The pier has a cast-in-place concrete deck working surface supported by treated-timber stringers, pile caps, and piles. Stringers are 6 by 12 inches and are spaced roughly 30 inches on center. In the areas of the railroad tracks, there are typical rail chords consisting of 10- by 18-inch timber stringers that support timber railroad ties. Pile caps are typically 14- by 14-inch timber with double caps in areas outside the rail stringer areas. Pile bents are braced using 4- by 10-inch treated-timber diagonal braces.

In 1997, a new mooring dolphin was constructed within the footprint of the deck at the river end of the pier.

In 2000, the southerly 12 feet of the deck was replaced with a new concrete apron. Pile caps were repaired/replaced and new piles were installed where required.

In 2004, a cantilever sheet pile cut-off wall was installed at the face of pier to allow for an increased dredge depth at the berth.

In 2013, a new shiploader tower foundation was installed near the joint between Berths 410 and 411.

The overall condition of the pier appeared to be fair with some of structural members exhibiting deterioration and water staining, including the diagonal bracing and pile caps. Lower brace to pile connections were deteriorated in many locations. Water intrusion from the deck appears significant.

Live loads have been restricted by the Port due to the condition of the structure. The primary uses of the pier appear to be as an access path to the mooring dolphin at the outboard end of the pier and as a berthing platform.

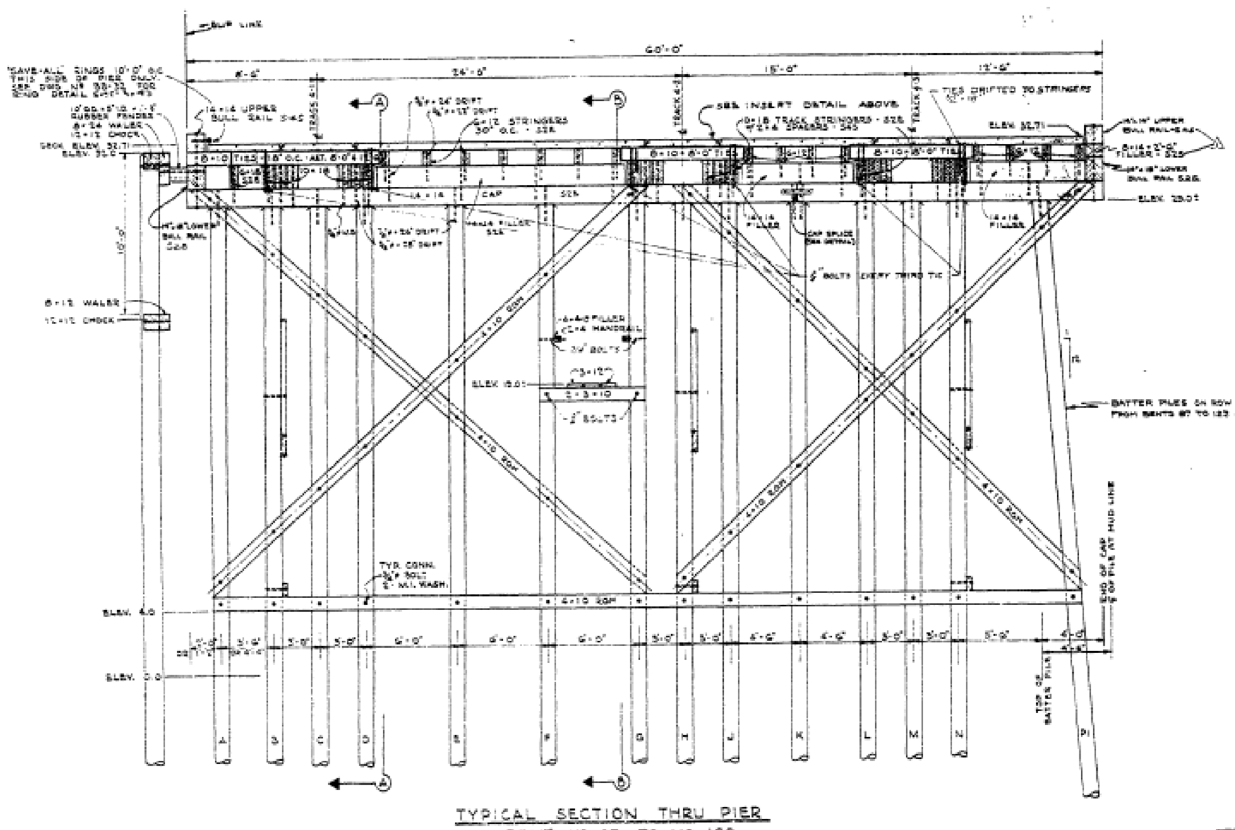

Figure 1 – Berth 410 typical pier section

Seismic Assessment and Estimated Damage

The seismic lateral system of the pier consists of a concrete deck and timber stingers that distribute the deck seismic forces to timber pile caps and then to timber diagonal braces attached to the plumb timber piles. Diagonal braces begin at the pile caps and extend approximately 29 feet below deck to a horizontal tie beam. Below the diagonal braces, seismic lateral forces are transferred to the mudline through flexure of the timber piles. The pier was originally designed for seismic lateral loads equal to 3.3 percent of its weight. Current seismic design codes require larger lateral seismic design forces, which can vary from roughly 10 percent of its weight for small earthquakes (OLE), to more than 30 percent of its weight for larger seismic events (DE).

In general, the pier has a complete lateral load path. However, the performance of the structure is vulnerable due to the age, condition and lateral-load resistance capabilities of the structural elements, particularly at the lower connection of the diagonal bracing to the piles. Additionally, potential post-earthquake soil settlement and soil lateral spreading displacements at Berth 410 are significant. Figure 2 below shows a typical Berth 410 pier cross section indicating the potential zone of soil lateral spreading.

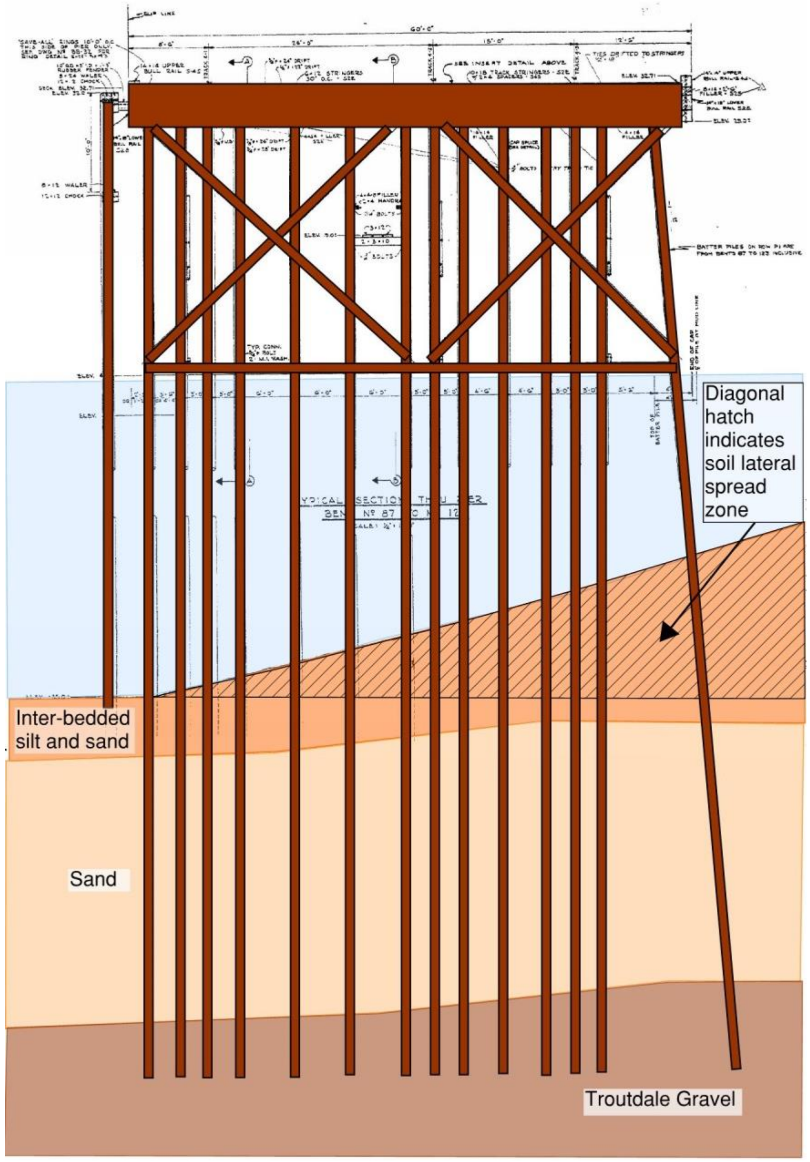

Figure 2 – Berth 410 typical pier section indicating zone of soil lateral spreading

For the OLE, soil lateral spread displacements were estimated at 2 feet. The pier likely has the ability to flex with these estimated soil spread displacements. The existing connection of Berth 410 pier to Berth 411 wharf was not detailed as a seismic joint. Because the two berth structures will shake differently during a seismic event, the lack of a seismic joint may cause them to pound against each other producing localized damage at the joint. In addition, pounding may also induce additional seismic demand on the lateral resisting system of the pier and on the mooring dolphin added in 1997.

It is not likely that the current function of the dock would be compromised in the OLE event and the mooring dolphin should still be accessible. We understand that the new shiploader tower foundation installed in 2013 was designed to current codes. We expect it to perform well at the OLE event; however, if the new tower foundation is not properly isolated from the seismic effects of the wharf, it may be subjected to much larger forces imposed on it by the adjacent wharf structure. A more detailed evaluation of the new tower foundation is recommended to better understand potential damage.

Following the OLE, damage to the pier structure is estimated to be minor to moderate and should be repairable. Damage could consist of cracks in the concrete deck, broken bracing members and piles, and localized deformations at bracing and pile connections. The bracing was intended for pier stability and to prevent individual pile buckling. Broken bracing elements and connections could reduce the overall gravity load carrying capacity.

At the CLE event, the seismic forces and resulting damage to the piles and bracing will be significantly greater than at the OLE. Soil lateral spreading was estimated at 5.5 feet, and the pier will not likely have the ability to flex and move with this amount of soil displacement. Expected damage consists of broken bracing elements, broken timber piles below the mudline, permanent deformations and partial collapse of portions of the pier. We expect that portions of the gravity load carrying capacity will be compromised and damaged beyond repair. Seismic forces from the pier could transfer to the mooring dolphin, possibly damaging the dolphin. Differential lateral movements between the pier structure and the conveyor tower foundation could compromise the function of the conveyor tower, depending on actual interface and connection details. We expect a post-earthquake inspection would result in closure of the structure.

At the DE event, soil lateral spread deformation was estimated to be more than 12 feet with seismic inertial forces greater than at the CLE. The pier structure is expected to have more significant damage and deformations than at the CLE, may experience partial or total collapse, and would be closed following post-event inspection.

Mitigation Measures

Due to the age of Berth 410, the current condition of the timber pier structure, and the extensive cost for soil mitigation to minimize lateral spreading hazards to the pier, seismic upgrades to the existing structure are not considered reasonable. One potential alternative is to replace and reconstruct the pier structure with a modern facility at the current Berth 410 location or at a new location. A replacement pier structure could consist of precast concrete deck panels with cast-in-place concrete pile caps and either prestressed concrete or steel pipe piles.

Other mitigation measures that could be considered to improve the expected performance of the existing structure include ground improvements and ongoing retrofit and replacement of timber members.

Repair Downtime and Cost

Repairs to the Berth 410 timber pier following the OLE would include crack repairs in the concrete deck, replacing and retrofitting bracing and pile connections, and replacing approximately 10 percent of the timber piles. After the OLE, the pier is not expected to have a significant downtime for repairs since the 1997 mooring dolphin would not require downtime, and access to the dolphin is anticipated to remain open. The repair costs are estimated at 15 percent of the cost of a new structure, approximately $1,800,000.

Following the CLE and DE events, the pier is expected to require complete reconstruction. Since Berths 410 and 411 would likely be reconstructed as a single berth, we estimate the replacement downtime for a new pier to be 26 to 38 months, with an associated cost of construction being $42,100,000.

Terminal 4, Berth 411

Structure Description and Condition

Berth 411 is part of the Kinder Morgan dry bulk facility and is a concrete wharf structure constructed in 1959. The wharf is used for the export of soda ash. The overall width and length is approximately 110 by 760 feet, respectively. The wharf supports a fixed shiploader, various rail tracks, and was originally designed to support a traveling unloading tower. The wharf is constructed with concrete including precast piles, cast-in-place pile caps, rail beams, and deck. The bent spacing is 30 feet on center and the outboard (outer 60 feet of the deck) support piles are 20-inch-square precast concrete and the inboard piles are 16-1/2-inch octagonal prestressed concrete. A timber pile bulkhead is located at the east end of the wharf, near the bottom of the river embankment, and extends south along the water edge.

In 2004, a cantilever sheet pile cut-off wall was installed at the waterside face of wharf to allow for an increased dredge depth at the berth to -45 feet.

In 2013, a new conveyor tower and support foundation were constructed at the west end of Berth 411. Construction drawings and calculations for the tower and support foundation were not readily available for this seismic assessment.

Based on the walk-through and review of the 2013 BergerABAM condition survey, the wharf structure is judged to be in fair condition, with minor damage to the piles, and cracking and spalls in the concrete pile caps and deck.

Photo 2 – Berth 411 typical concrete piles and pile caps arrangement

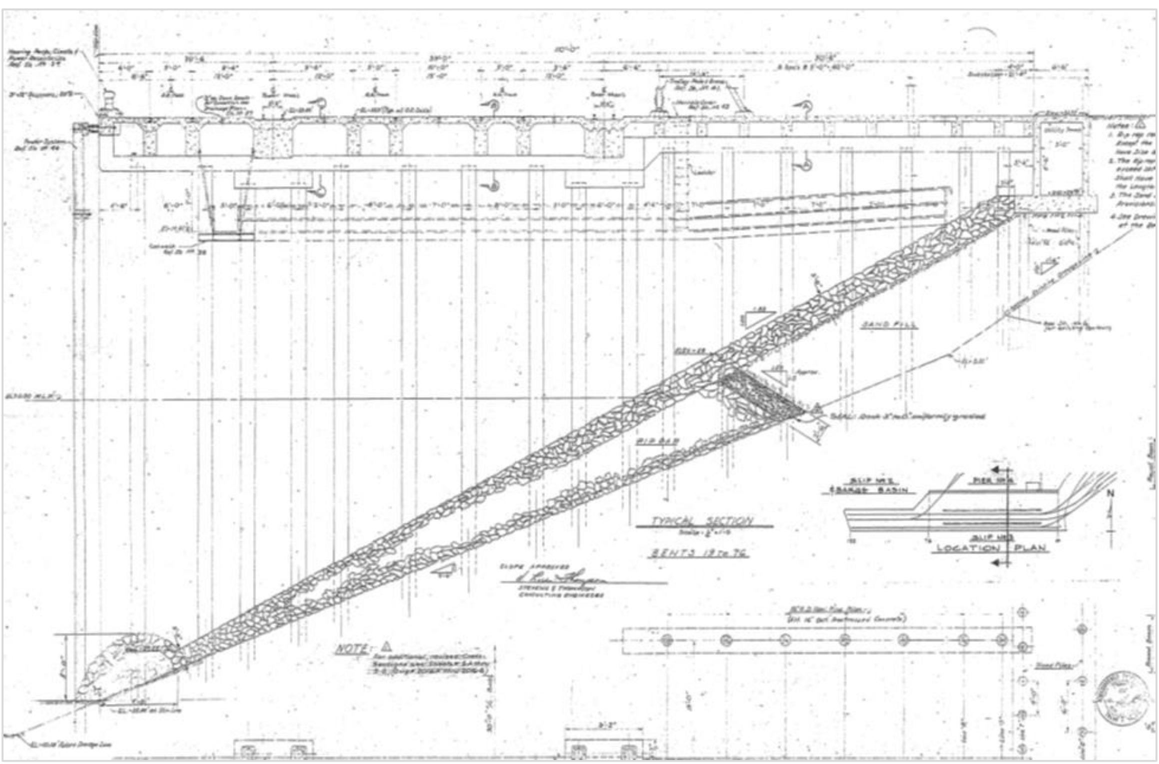

Figure 3 – Berth 411 typical wharf section

Seismic Assessment and Estimated Damage

The seismic lateral system of the wharf consists of a concrete deck and beams distributing the seismic loads to cast-in-place concrete pile caps. The pile to pile cap connection is detailed as a pin-type connection, which indicates this structure was designed as a cantilever pile system that transfers inertia load of the deck to the mudline through flexural stiffness and embedded fixity of the piles. The wharf was originally designed for seismic lateral loads of 3.3 percent of its self-weight. Current seismic design codes require larger lateral seismic design forces, which can vary from roughly 10 percent of its weight for small earthquakes (OLE), to more than 30 percent of its weight for high seismic events (DE).

The sheet pile toe wall installed in 2004 was designed with an equivalent horizontal surcharge for seismic conditions at a 475-year event assuming a 0.15g earthquake acceleration for the modeling.

Post-earthquake soil settlement and soil lateral spreading displacements are estimated to be significant at Berth 411. Figure 4 below shows a typical Berth 411 cross section indicating the potential zone of soil lateral spreading.

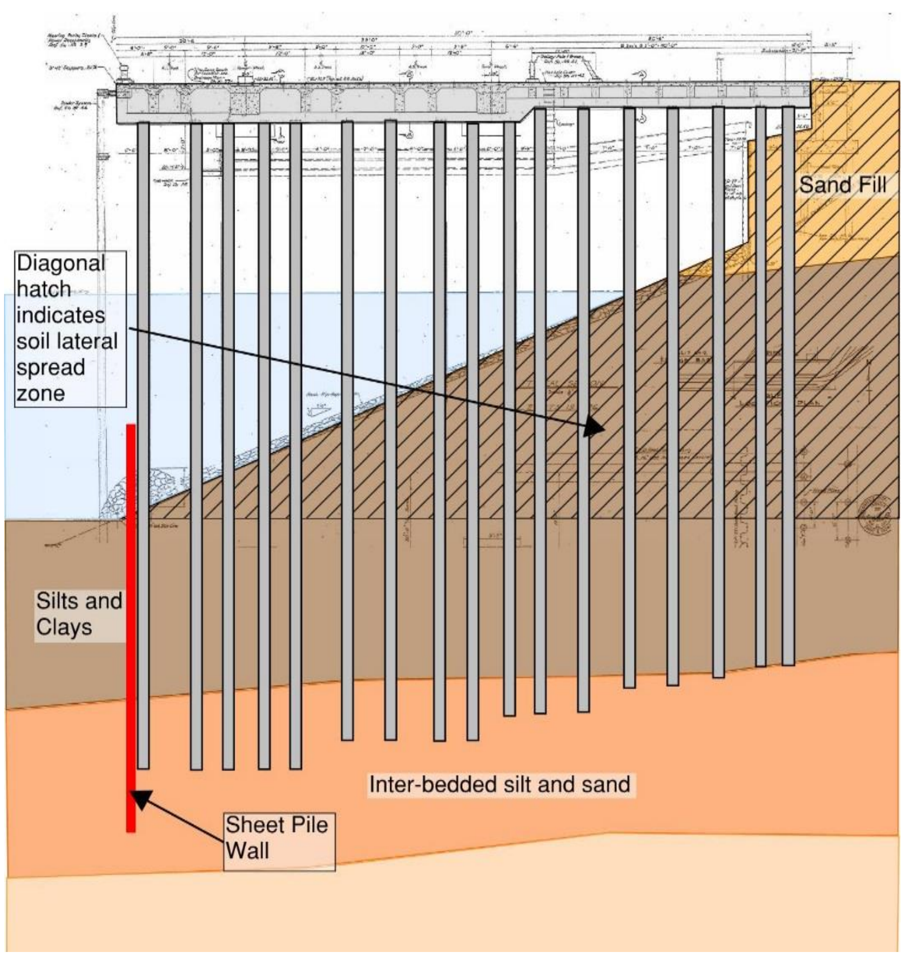

Figure 4 – Berth 411 typical wharf section indicating zone of soil lateral spreading

Concrete wharf structures, such as Berth 411, with a large number of slender plumb piles, have a redundant load path for vertical loads and typically perform adequately during small to moderate seismic events. At the OLE, the estimated level of damage is expected to be minor to moderate and should be repairable. The damage may include minor cracks of the concrete deck, cracks at the pile caps, and damage to the shorter concrete piles at the land side near the top of slope. Some damage to the timber piles under the bulkhead should be expected. Soil lateral spreading at the OLE were estimated at 0.5 foot. It is likely that the wharf is flexible enough to accommodate the soil lateral spread displacements. We understand that the new ship loader tower foundation installed in 2013 was designed to current codes. We expect it to perform well at the OLE, provided that the foundation is properly isolated from the seismic behavior of the wharf. If this is not the case, the shiploader tower foundation may be subjected to much larger forces imposed on it by the adjacent wharf structure. Some damage to the conveyor bridge and tower connections could be expected however a more detailed evaluation of the new tower foundation and of the materials handling system is recommended to better understand the potential damage.

At the CLE, the seismic forces and resulting damage to the structure will be significantly higher than at the OLE. Soil lateral spreading was estimated at 3 feet and will impose a large lateral surcharge on the concrete piles. No seismic joint was provided where Berth 411 connects to the adjacent Berth 410 pier. The two adjacent structures are expected to pound against each other during a seismic event. We expect damage would be cracks and spalling at the joint pier to wharf interface, differential separation between the pier and wharf potentially causing a localized collapse of the deck at the joint, cracks in the concrete deck and pile caps, and yielding of the connection of the plumb piles near the pile caps and the piles below the mudline. Connections to utilities at the ends of the dock may be severed due to the movement of the structure and displacement of surrounding ground. Gravity load carrying elements could be compromised by the CLE, and post-earthquake deflections could leave the wharf deck out of plumb by up to 3 feet.

The DE forces and soil lateral spreading deformations exceed those from the CLE event. We expect the wharf structure to exhibit more significant damage than that caused by the CLE. Repair would not likely be a reasonable option following the DE.

Mitigation Measures

Due to the age and condition of Berth 411 and the expected damage at the CLE and DE events, the most likely course of action for the facility would be to replace and reconstruct the wharf structure in lieu of a structural upgrade and ground improvement program. A replacement pier structure could consist of precast concrete deck panels with cast-in-place concrete pile caps and either prestressed concrete or steel pipe piles. We understand from discussions with the Port that in a reconstruction condition, Berth 410 and 411 would be reconstructed together.

Apart from the wharf structure, to improve the performance of the conveyor tower and shiploader foundations during the OLE and CLE, the wharf deck could be cut around the foundations in order to seismically isolate them from the rest of the wharf. The tower structure and supporting foundation could be strengthened to resist lateral spreading displacements during a seismic event.

Repair Downtime and Cost

Repairs to Berth 411 following the OLE is expected to be limited to repairing cracks and spalls in the deck surface, pile caps, and piles. Repair or replacement of some of the shorter plumb piles near the top of the slope may also be required. The repair downtime is estimated at 5 to 8 months, and repair costs are estimated at 15 percent of the cost of a new structure, approximately $5,200,000.

Following the CLE and DE events, the pier (with the exception of the shiploader tower foundation) is expected to require complete reconstruction. The replacement downtime for a new combined Berth 410 and 411 wharf is expected to be 26 to 38 months, and the associated cost of construction is $42,100,000.

Terminal 5, Berth 501

Structure Description and Condition

Berth 501 serves the grain terminal and is operated by Columbia Grain, Inc. Berth 501 is a hybrid pier structure constructed in 1974. The pier structure has an overall length of approximately 610 feet and is located approximately 350 feet from shore and consists of three, freestanding, 55.67-foot-diameter sheet pile cell structures spaced 150 feet apart. The cell structures are back filled with compacted gravel and support concrete cap slabs placed on top of the cells providing a working dock surface. Each of the cell structures supports towers and grain conveying equipment. This equipment is founded on piles driven within each cell structure. The cells are interconnected by pile-supported aprons roughly 16 feet wide by 60 feet long. Pile-supported aprons extend upstream and downstream from the east and west cells, respectively, connecting with a pair of mooring dolphins. The aprons are constructed with a concrete deck and cast-in-place concrete pile caps supported on timber piles.

The dock structure is connected to shore at the upstream end by a trestle and retractable sliding bridge section. The trestle is constructed of pipe piles and steel members. The retractable bridge is also of steel construction.

Shoreward of the center pile cell are pipe pile-supported concrete caps that support the grain conveying equipment.

The general condition of the pier structure was not verified.

Photo 3 – Berth 501 dock and shiploader towers general arrangement

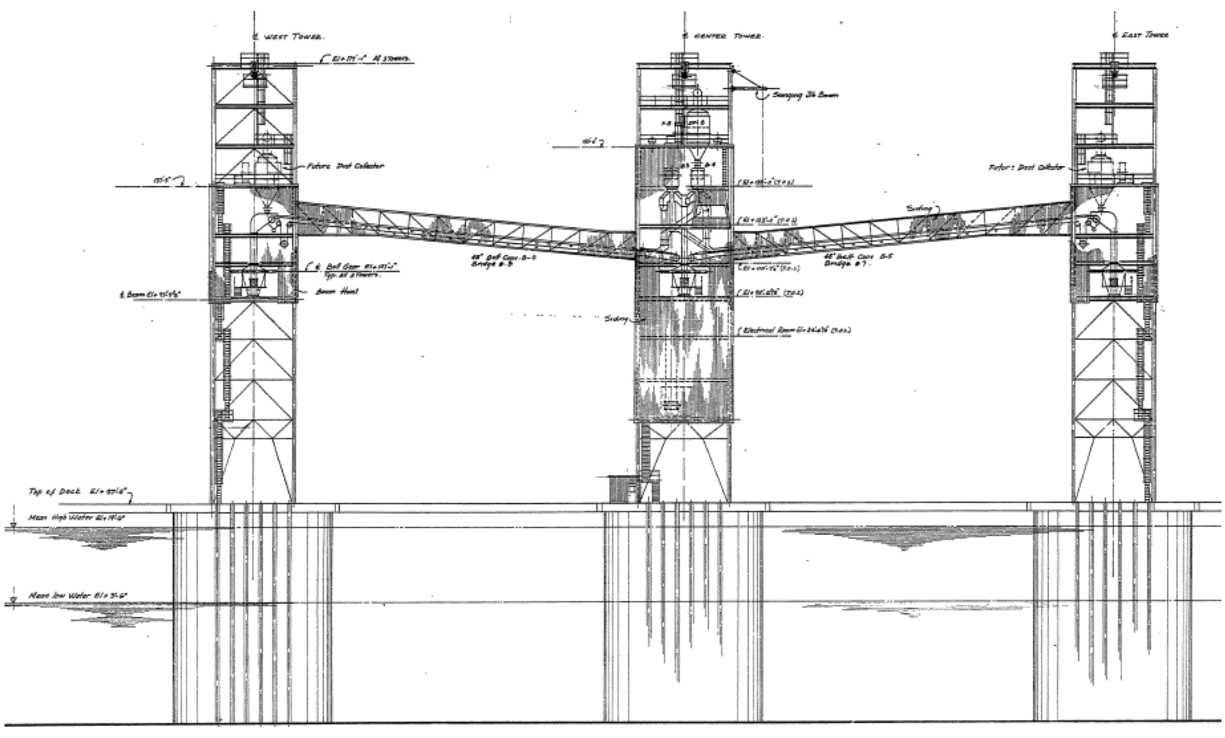

Figure 5 – Berth 501 cell structures and shiploader towers elevation

Seismic Assessment and Estimated Damage

Original seismic design criteria were not available for the dock structure; however, for a dock structure constructed during the mid-1970s, we assume that the lateral seismic design forces would be significantly less than the required design forces of current codes. The seismic lateral force-resisting system of the dock is provided by the three separate sheet pile cell structures, where the sheet pile and contained gravel backfill behave as a gravity-based structure. The apron longitudinal seismic lateral forces are transferred to the cell structures and transverse seismic forces are transferred to the piles supporting the aprons. The apron piles transfer seismic forces down to the mudline through flexural bending.

The shiploader towers at each cell structure are individually supported by concrete pile caps and 16-inch concrete octagonal piles driven inside the sheet pile cells. Differential displacement between adjacent cell structures is expected to be small; however, due to the large mass and stiffness of each cell structure, any differential displacement may impose very large seismic forces into the aprons depending on connectivity to the cells. The aprons may not have been designed to transfer these forces. The shiploader towers consist of multilevel steel-braced frames with a truss moment frame at the lowest level. The truss moment frame and connections to the dock could be vulnerable to localized yielding.

Post-earthquake soil settlement and soil lateral spreading displacements are estimated to be significant at Berth 501. Because the walls of the cell structures are constructed of steel sheets, they may be vulnerable to soil spreading surcharge loads potentially leading to localized bending or buckling of the sheets. Figure 6 below shows a typical Berth 501 cross section indicating the potential zone of soil lateral spreading.

Closer to shore, a bridge tower supports the conveyers for the shiploading towers and is founded on an isolated concrete dolphin on steel piles. The dolphin is highly susceptible to the lateral spreading displacements and could undergo large, permanent deformations after the earthquake. Conveyor bridges span from the tower to the shiploader towers, and any deformation of the conveyor tower and bridges could result in the transfer of large seismic forces.

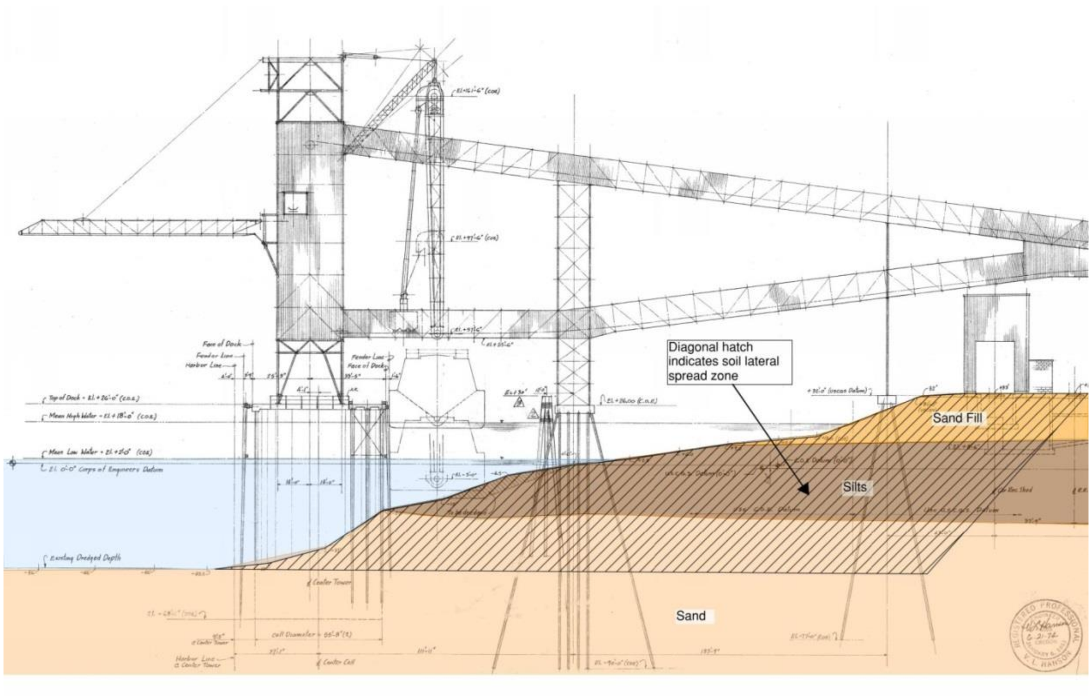

Figure 6 – Berth 501 dock section indicating zone of soil lateral spreading

At the OLE, the level of damage is expected to be significant with extensive repairs required. Soil lateral spreading displacements are estimated at 2.5 feet and may fracture and fail the timber piles supporting the aprons. The cell structures may remain stable and resist the soil lateral spread forces at the OLE; however, some localized deformation of the sheet pile walls may occur. The isolated conveyor tower dolphins may displace up to 2.5 feet due to the soil lateral spreading. This amount of displacement would likely damage the conveyor towers, conveyor bridges, and shiploader towers and possibly collapse parts of the conveyor systems.

At the CLE and DE events, the dock is expected to exhibit significant damage caused by the seismic inertial forces and the estimated 7 to 8 feet of soil lateral spread displacement. The cell structures, aprons, and trestle would be exposed to large lateral forces from the slope movement.

Mitigation Measures

Mitigation measures to increase the performance of the Berth 501 structures to survivability at the 475-year event would include soil improvement using stone columns installed on the river embankment, around the approach trestle abutment, and possibly within the cellular structures. Given potential permitting constraints, these soil improvements may only be feasible above the ordinary high water mark. The dock structure, including the cellular structures would remain vulnerable to a slope failure/lateral spreading event. The existing conveyor bridge tower dolphins should be strengthened to resist lateral spreading and seismic inertial forces at the CLE event by adding new piles. Conveyor bridge connections at towers should also be strengthened to prevent the bridges from pulling away from the tower. Connections of the shiploader and conveyor towers to the dock structure should be retrofitted to achieve the strength needed to provide ductile behavior in the tower frame elements. The cost of mitigation measures to achieve survivability at the CLE is estimated at $19,500,000.

Repair Downtime and Cost

Repair downtime and the cost of repair for Berth 501 after an OLE is estimated to be 12 to 16 months at a cost of $8,500,000.

Without mitigation, following the OLE and CLE, the pier is expected to be significantly damaged due to the fracture of plumb and batter piles below the mudline caused by seismic inertial forces and the soil lateral spreading. Permanent deformations and settlement of the dock, cell structures, and towers could make repairs impractical. The replacement downtime for a new dock is expected to be 22 to 34 months with an associated cost of $27,700,000. This cost does not include reconstruction costs of the conveyors, towers, and other mechanical equipment on the dock.

Terminal 5, Berth 503

Structure Description and Condition

Berth 503 serves the mineral bulk terminal and is operated by Portland Bulk Terminals. The structure has a concrete deck, concrete pile caps and beams, and plumb concrete piles. Battered steel pipe piles are located at each fender. The pier was constructed in 1982 and is approximately 830 feet long by 44 feet wide. The structure arrangement consists of two continuous longitudinal pile caps (parallel to river) that support the rails for the traveling shiploader crane. Transverse tie beams are spaced approximately every 13 to 14 feet on center. Partial-width infill precast deck panels are provided for supporting vehicle access (14 feet wide). At three discrete locations, a heavy-duty deck has been provided across the full width for locating maintenance equipment to service the shiploader and conveyor system. Piles include large-diameter battered steel pipe piles (42-inch-diameter at the fender locations and 36-inch-diameter at the trestle) and hollow 24-inch octagonal precast concrete piles. Integral with the deck structure are six fender panels, spaced roughly 145 feet apart. A trestle that connects the pier to shore is provided at the downstream end of the pier.

The overall condition of the pier appeared to be satisfactory. Several concrete structural members exhibited cracking and spalling with some rust staining.

Photo 4 – Berth 503 dock and trestle bridge

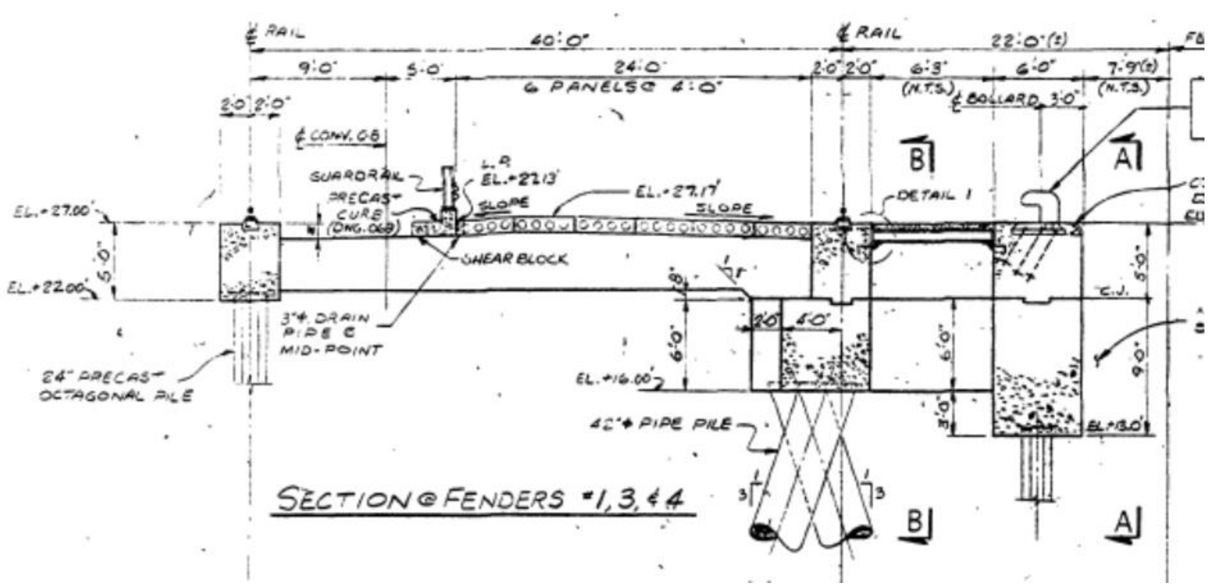

Figure 7 – Berth 503 typical dock section

Seismic Assessment and Estimated Damage

Original seismic design criteria were not available for the dock structure. The lateral force-resisting system of the dock consists of a concrete deck that distributes the deck inertia forces to supporting concrete pile caps below the deck. The pile caps and supporting piles provide lateral stability of the structure and allow the seismic forces to be transferred to the mudline through flexural stiffness of the concrete piles and axial stiffness of the steel batter piles.

Soil lateral spreading displacements are expected to be significant at Berth 503. Figure 8 below shows a typical Berth 503 cross section indicating the potential zone of soil lateral spreading.

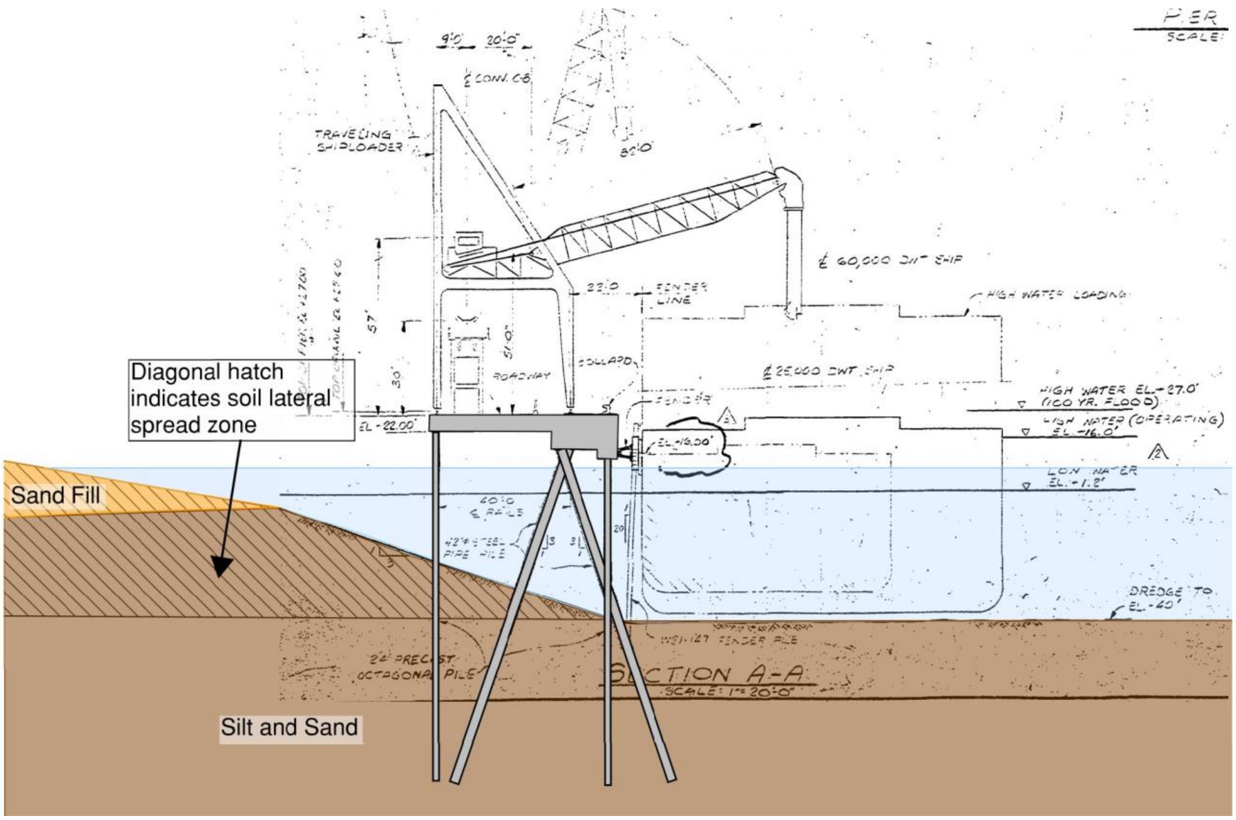

Figure 8 – Berth 503 dock section indicating zone of soil lateral spreading

At the OLE, the dock structure is expected to perform adequately. A lateral spreading event is triggered at the OLE with estimated soil displacements of approximately 1 foot. The piles are expected to resist the lateral spreading displacements and forces. We expect minor to moderate repairs to be required for the deck, piles, and pile caps. Closer to shore, the approach trestle may be significantly damaged due to the effects of lateral spreading.

A structural capacity assessment of the dock at a 475-year earthquake was conducted in 2012 by Hatch Mott MacDonald. The assessment of the dock structure considered a U.S. Geological Survey response spectrum and an Oregon Structural Specialty Code 2010 Site Class D, the results of which indicated demand capacity ratios of structural elements were near, or just above, capacity for the 475-year event. The flexural capacity of the longitudinal concrete beams was shown to have been exceeded by 25 percent. The assessment indicated that the dock structure would be "damaged but repairable." Effects of liquefaction and lateral spreading were not considered.

A separate assessment was conducted by WorleyParsons in 2014. The results of the seismic analysis indicates that with a new shiploader installed at a 475-year earthquake, no lateral load resisting elements had a stress increase of more than 10 percent above the original dock design values. The WorleyParsons report indicates that the seismic analysis did not include kinematic soil loading on the dock from lateral spreading; therefore, the susceptibility of the wharf to liquefaction-induced soil failures was not assessed. The report highlights that in the past, slope failures have led to "excessive damage on marine structures in the past and therefore carry inherent risk of economic loss should they be triggered at this site." Once the impact of soil movements due to lateral spreading are included (on the order of 7 feet at the 475-year level), more significant damage to the structure is expected. Given this background, the structure may be vulnerable to significant damage at the CLE and DE events.

Mitigation Measures

In order to meet a CLE-level earthquake with significant, but repairable damage, a ground improvement program would be conducted along the shoreline, piles and connections would be strengthened, and the concrete beams would be strengthened. Assuming ground improvements and a structural retrofit cost of 30 percent of a new structure, the estimated cost of a partial retrofit is $13,100,000.

Repair Downtime and Cost

Repair downtime estimates and cost for repair of Berth 503 following the OLE is estimated at 5 to 8 months at a cost of $9,000,000. The repair cost is estimated as 30 percent of the cost of a new structure.

Following the CLE and DE, the berth could be significantly damaged due to the magnitude of the soil movements compounded with the effects of the inertial loading. The replacement downtime for a new dock is expected to be 26 to 38 months at a replacement cost of $37,800,000.

Terminal 6, Berth 601

Structure Description and Condition

Berth 601 is an automobile import facility constructed in 1989 featuring a floating dock. Berth 601 consists of two steel pontoons connected together to form a floating dock roughly 450 feet long by 101 feet wide. The working surface is asphalt concrete paving. The dock is held in place by four breasting dolphins and a series of eight catenary wire rope mooring lines that are anchored to four mooring dolphins. The dock is connected to shore by a steel transfer span and pile-supported trestle. The transfer ramp is hinged to accommodate water level fluctuations.

The overall condition of the dock and approach trestle appeared to be good.

Photo 5 – Berth 601 trestle and access ramp

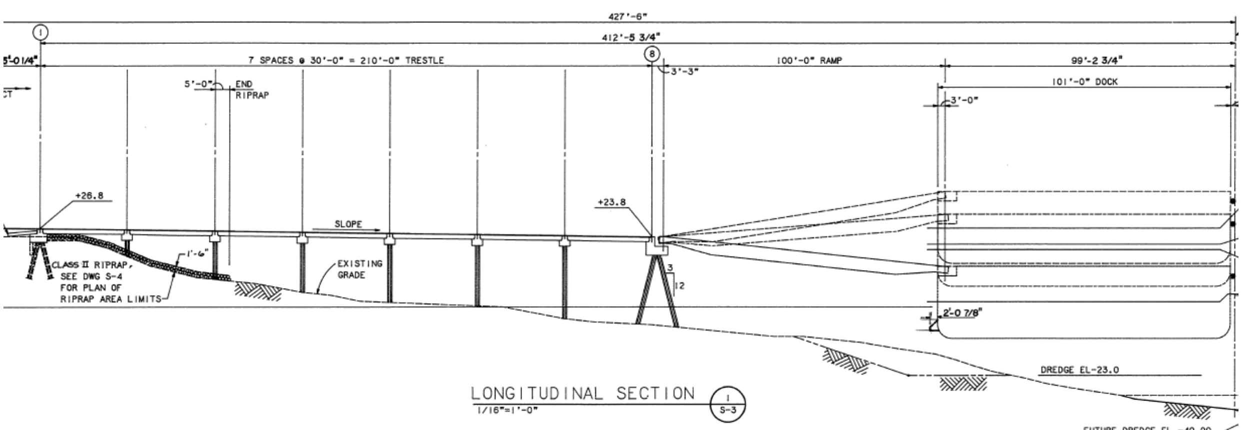

Figure 9 – Berth 601 trestle longitudinal section

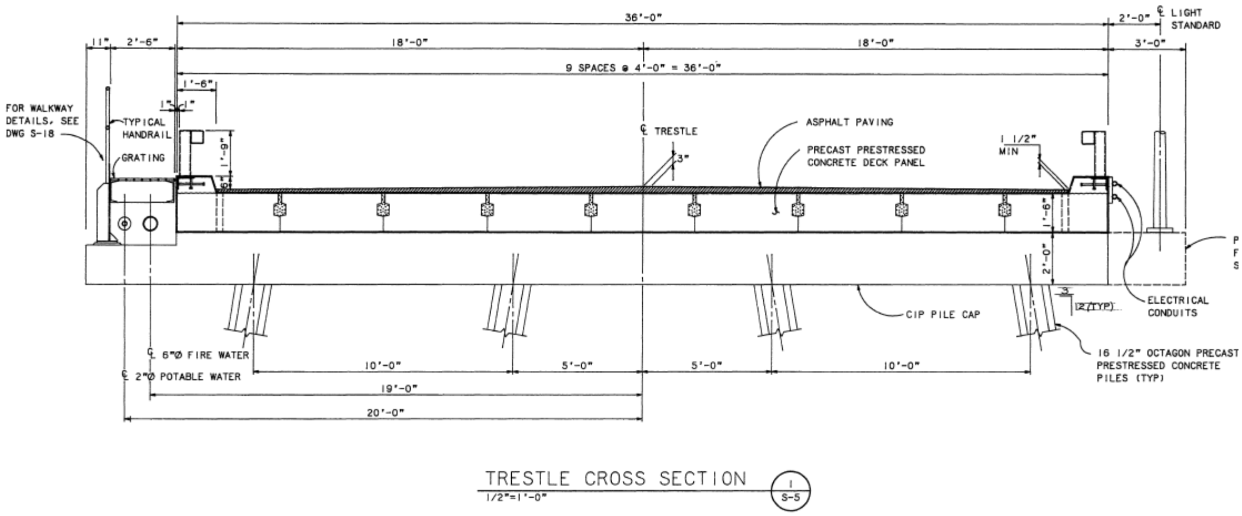

Figure 10 – Berth 601 trestle typical cross section

Seismic Assessment and Estimated Damage

The lateral force system of the floating dock consists of breasting dolphins and a catenary wire rope mooring lines that are anchored to mooring dolphins. The wire ropes and dolphins act to restrain the pontoons during a seismic event.

The approach trestle is supported by a shore side abutment and bents spaced at 30 feet on center. Transverse seismic forces of the trestle are transferred to the bents and batter piles and longitudinal seismic forces are resisted by the shore side abutment. An articulating ramp spans between the trestle and the floating pontoons. The ramp is supported by a roller bearing at the pontoon end and is fixed at the hinge bent of the trestle.

The trestle structure was originally designed to the 1985 Unified Building Code, Zone 2 criteria. The design lateral seismic force for the approach trestle was approximately 11 percent of its self-weight, which is approximately equal to design lateral forces at the OLE. Current seismic design codes require larger lateral seismic design forces at the DE, which can be more than 30 percent of its seismic weight.

The floating pontoon components are not expected to experience significant damage from seismic events because seismic inertial forces are highly dampened.

Soil lateral spreading displacements are expected to be significant at Berth 601. Figure 11 below shows a longitudinal section at Berth 601 approach trestle indicating the potential zone of soil lateral spreading.

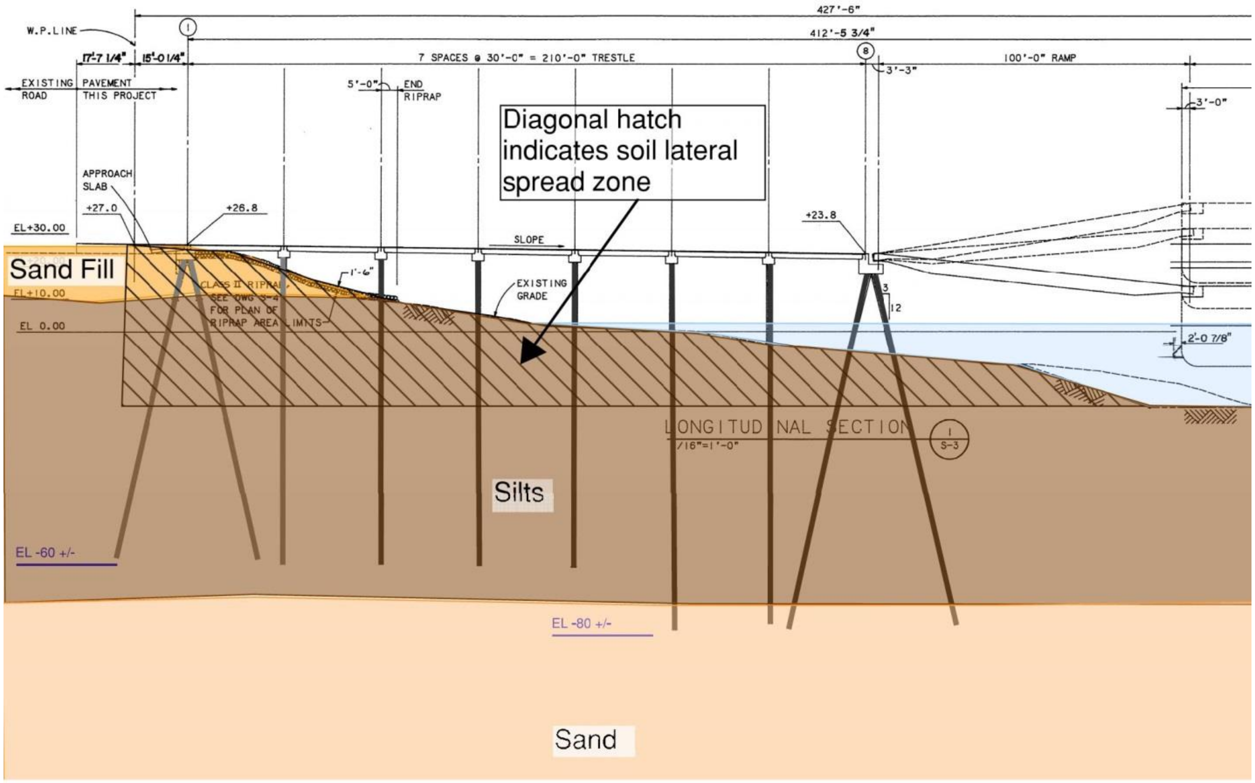

Figure 11 – Berth 601 trestle longitudinal section indicating zone of soil lateral spreading

Soil lateral spread displacements were estimated at 4 feet at the OLE and 17 feet for the CLE. Due to these expected displacements, the trestle and mooring dolphins may be significantly damaged at the OLE.

Mitigation Measures

Mitigation measures for Berth 601 could include soil improvement using stone columns installed around the approach trestle bents and abutment. Retrofit of the concrete trestle for the inertial loading at the CLE and DE may require installation of new piles at each bent. The estimated mitigation cost to retrofit the trestle is $4,500,000. Structural mitigation costs (not including ground improvements) for installing additional piling and improving connections is estimated at 30 percent of a new structure.

Repair Downtime and Cost

The trestle and landward dolphins could be significantly damaged at the OLE, CLE, and DE events for the current, unimproved condition due to significant lateral spreading displacements. The replacement downtime for a new trestle and dolphin is expected to be 15 to 21 months with an associated cost of construction of $13,300,000. This estimate assumes the floating pontoon could be salvaged and reused.

Terminal 6, Berths 604/605

Structure Description and Condition

Berths 604 and 605 are 1,800 lineal feet of sand-filled cellular sheet pile structures constructed in 1974. Together with Berth 603, these berths serve as the Port's container terminal. In 1994 to 1995, Berths 604 and 605 were structurally modified to accommodate new container cranes. In 2006, a 100-foot long sheet pile wall was installed in front of Berths 604 and 605 to control ship scour. In 2011 and 2012, partial seismic upgrades consisting of jet grouted columns within the main cells and pile arcs and a combination of jet grouted columns and stone columns landward of the main cells was completed on an 800-foot portion of the wharf. The working surface is asphalt concrete paving. At the face of the dock is a combined steel pile/timber pile fender system.

The overall condition of the wharf was considered satisfactory to good in 2013.

Photo 6 – Berth 604/605 cellular sheet piles

Seismic Assessment and Mitigation Measures

The work completed on Berths 604 and 605 in the mid-1990's included a design basis earthquake with a 10 percent probability of exceedance in 20 years (approximately 190-year return period). Based on the assessments completed prior to the seismic upgrades made in 2011 and 2012, it was estimated that the wharf in its unimproved state could survive up to a seismic event with a 50-year return period. The partial upgrades completed improved the 800-foot long portion to survive the 200-year earthquake. The repair time for this event was estimated at 4 to 6 months. A benefit/cost study completed in 2012 by GeoEngineers assessed the potential benefits of a seismic upgrade to the entire 1,800-foot long wharf to meet the 475-year earthquake with repairable damage (GeoEngineers, 2012). Figure 12 presents the estimated reduction in annualized seismic loss for the partial upgrade scenario.

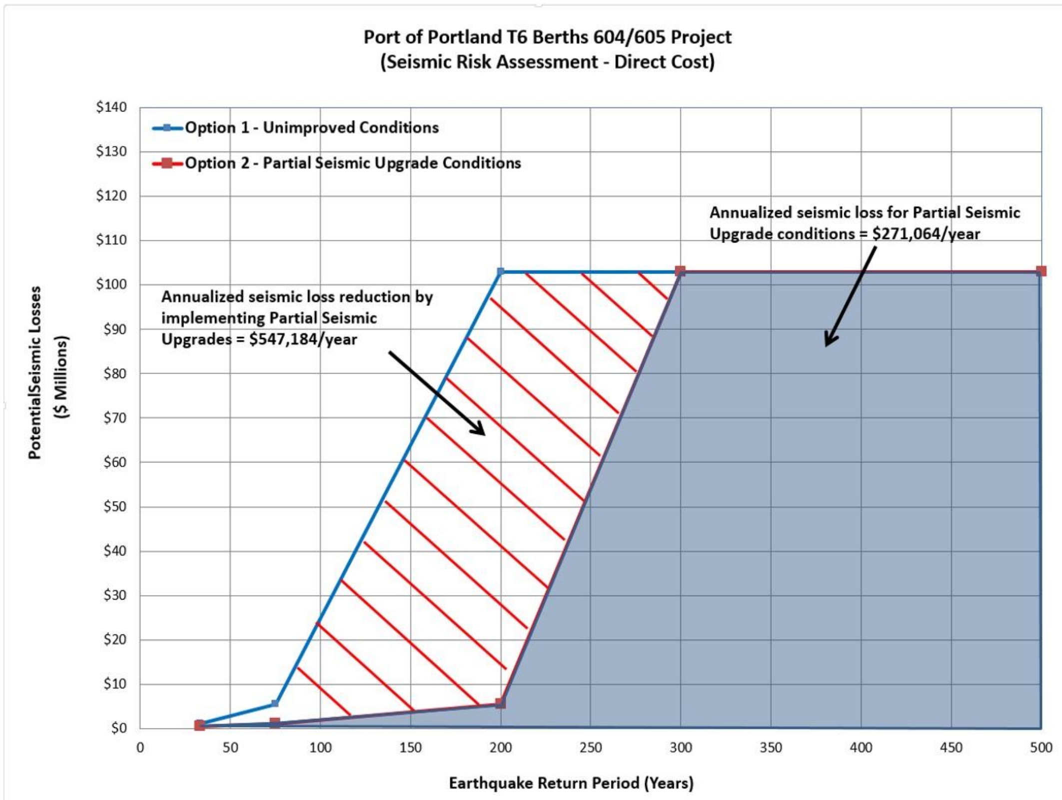

Figure 12 – Berth 604/605 partial seismic retrofit annualized seismic loss reduction

CONCLUSIONS

The assessment of the prioritized assets indicates the waterfront structures at Berths 410, 411 and 503 will perform adequately at the OLE. Berths 501 and 601 are estimated to have significant damage at the OLE. At the CLE and above, we conclude that all waterfront structures are vulnerable to the effects of large estimated soil lateral spreading displacements and may experience significant damage. We recommend a more rigorous analysis in a subsequent phase of the seismic risk assessment program to further evaluate risk and potential mitigation measures.

A summary of the study findings is presented in Appendix A.

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Port of Portland
Marine Facilities Risk Assessment
Portland, Oregon

Appendix A
Damage, Downtime, and Cost Table

Approximate Benefit/Cost Analysis, Marine Facilities, Port of Portland

ImageCat, Inc. **11/5/2014** rev'd 12/18/14

				On Port Soils				Status quo (As-is) Structure Damage and Downtime				Structure Damage and Downtime with Partial Retrofit						Structure Damage and Downtime for New Replacement Structure			
Berth	Replacement Cost New	Annual Downtime Cost (Port Only)	Annual Downtime Cost (region)	Earthquake Return Period	PGA	Ss	S1	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Partial Retrofit Measure	Cost of Partial Retrofit Measure	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)
410/411	$42,100,000	$ 3,400,000	$ 90,000,000	72 Years	.09	.17	.11	410:Minor to moderate repairs; deck repairs, bracing and pile connections; replacement of 10% of piles; 411: deck repairs, pile and pile cap repairs	$7,000,000	6	5 to 8	No feasible partial retrofit alternative to meet 475-year EQ. Reconstruction is the mitigation	N/A	(see columns to right)				Minimal damage (est. 5%)	$1,500,000	0	0 to 1
				475 Years	.25	.50	.42	Total reconstruction	$42,100,000	32	26 to 38							Controlled and repairable damage (est. 30%)	$9,100,000	3	2 to 4
				975 Years	.30	.62	.55	Total reconstruction	$42,100,000	32	26 to 38							Reconstruction	$42,100,000	32	26 to 38
501	$27,700,000	$ 3,000,000	$ 180,000,000	72 Years	.09	.17	.11	Moderate to significant repairs to structures (est. 50%)	$8,500,000	14	12 to 16	Significant ground improvements around dock and in cells; pile/pilecap strengthening; strengthen conveyor tower supports with new batter piles; strengthen conveyor system tower and bridge connections.	$19,500,000	Minor damage (est. 10%)	$1,700,000	3	2 to 4	Minimal damage (est. 5%)	$900,000	0	0 to 1
				475 Years	.25	.50	.42	Total reconstruction	$27,700,000	28	22 to 34			Significant, repairable damage (est. 50%)	$8,500,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$5,100,000	3	2 to 4
				975 Years	.30	.62	.55	Total reconstruction	$27,700,000	28	22 to 34			Reconstruction	$24,100,000	28	22 to 34	Reconstruction	$27,700,000	28	22 to 34
503	$37,800,000	$ 2,500,000	$ 98,000,000	72 Years	.09	.17	.11	Minor to moderate repairs including deck repairs, pile and pile cap repairs; significant trestle reconstruction	$9,000,000	6	5 to 8	Ground improvement program along shoreline, strengthen piles/connections, strengthen concrete beams	$13,100,000	Minor damage (est. 10%)	$3,000,000	3	2 to 4	Minimal damage (est. 5%)	$1,500,000	0	0 to 1
				475 Years	.25	.50	.42	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Significant, repairable damage (est. 50%)	$15,000,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$9,000,000	3	2 to 4
				975 Years	.30	.62	.55	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Reconstruction	$35,750,000	32	26 to 38	Reconstruction	$37,800,000	32	26 to 38
601	$13,300,000	$ 2,000,000	$ 18,000,000	72 Years	.09	.17	.11	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21	Ground improvements around the approach trestle and abutment, strengthen trestle piles/connections at batter pile bents, retrofit dolphins	$4,500,000	Minor damage	$1,100,000	1	0 to 2	Minimal damage	$600,000	0	0 to 1
				475 Years	.25	.50	.42	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Significant, repairable damage	$5,400,000	6	4 to 8	Controlled and repairable damage	$3,300,000	3	2 to 4
				975 Years	.30	.62	.55	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Reconstruction of a new trestle and dolphins	$12,700,000	18	15 to 21	Reconstrcution of a new trestle and dolphins	$13,300,000	18	15 to 21
604/605	$100,000,000	$ 11,000,000	$ 120,000,000	72 Years	.09	.17	.11	Minor damage, possible rail reconstruction (est. 10%)	$10,000,000	3	1 to 4	Full seismic upgrades described in GeoEngineers report that targets 500-year EQ for repairable damage	$15,300,000	Minor damage (est. 5%)	$5,000,000	1	0 to 2	Minimal damage (est. 5%)	$5,000,000	0	0 to 1
				475 Years	.25	.50	.42	Reconstruction	$100,000,000	30	28 to 32			Significant, repairable damage (est. 15%)	$15,000,000	3	2 to 5	repairable damage (est 10%)	$10,000,000	3	2 to 4
				975 Years	.30	.62	.55	Reconstruction	$100,000,000	30	28 to 32			Reconstruction	$100,000,000	30	28 to 32	Reconstrcution	$100,000,000	30	28 to 32

Notes:
1. Materials handling equipment, inlcuding conveyors and towers not included
2. Tenant-owned infrastructure cost and construction time is not included
3. Berth 410/411 is combined - new structure is assumed to be an 850'x80' dock with access trestle and two mooring dolphins
4. At B601, reuse of pontoons is assumed
5. Acquisition of permits for in-water construction is assumed to be 12 months for new construction.
6. Ground improvement costs have been adjusted in cases where there is reconstruction after mitigation
7. Ground improvements made during partial retrofit at 501, 503, and 601 will not be fully reconstructed post-950 year event. Assumed 50% of ground improvement replacement cost.

Tom Wharton: Regional economic numbers do not include Port numbers – these are separate and distinct

Correction by Scott McMahon, 10/21/2014 -- repair cost 410/411 for 72-year scenario

Appendix 5 – Seismic Risk and Benefit-Cost Analyses

(ImageCat, Inc.)

ImageCat Documentation

Seismic Risk Analysis and Benefit/Cost Analysis

1 Overview

ImageCat, Inc. of Long Beach, California served as the seismic risk consultant to the Port of Portland on this project, working closely with other project participants. Project members from ImageCat included Dr. Craig E. Taylor, William Graf, P.E., Yajie Lee and Charles Huyck, along with other members of ImageCat's staff.

The facilities to be modeled include PDX buildings, marine facilities (T4, T5 and T6). ImageCat did not evaluate the HIO facilities. ImageCat's software modeling system (SeismiCat) was used for the PDX buildings, with consideration of the business interruption impacts of runway damage. Simple, spreadsheet-based methods were used with the marine facilities and runways.

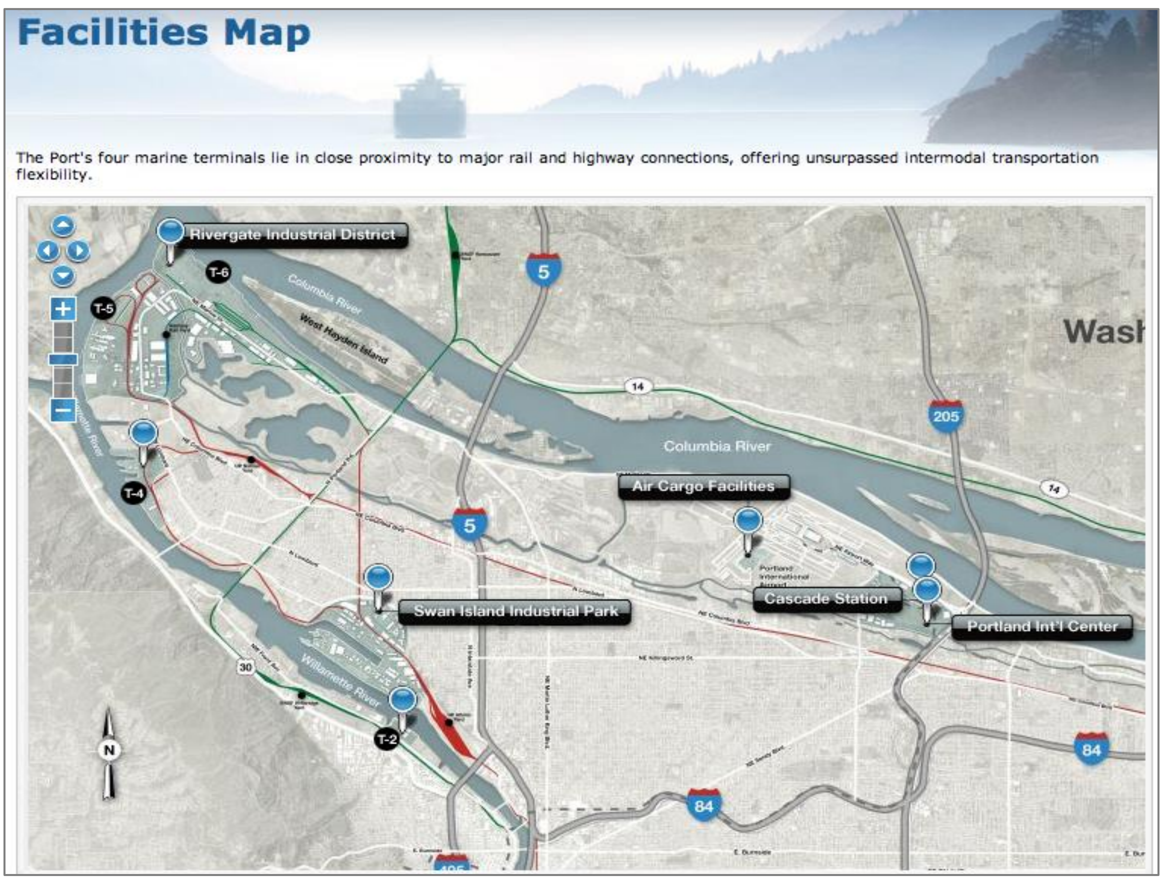


Figure 1 – Port of Portland Facilities

ImageCat's scope for this study included the following:

1) ImageCat made available the SeismiCat online seismic risk management system for use by KPFF in modeling the buildings at Portland international Airport (PDX). KPFF modeled the selected buildings with and without seismic retrofits, as described in Section 5. The outputs include level of building damage, expressed as a fraction of the building value, and the expected downtime for repair of the earthquake damage. The models include damage from shaking and damage from settlements caused by soil liquefaction.

2) ImageCat took the building-by-building modeling information as modeled by KPFF from the SeismiCat online system and imported it to the SeismiCat Multi-site tool for portfolio risk assessment. Here, the building replacement values and revenue loss rates associated with each building were used to obtain consequences in financial terms (dollars) as well as downtime. Of particular importance was input from the Port for regional impacts that would be caused by loss of function of Port facilities. These regional impacts are much larger than the impacts to the Port itself.

Facility	Asset	Rank	Minimum Replacement Value[1] (Insurance Coverage)	Engineering Estimated Total Replacement Cost[7]	Engineering Estimated Time to Rebuild[2] (Years)	Estimated Annual Port Revenue[3]	Estimated Impact to Port Revenue[4]	Estimated Annual Regional Economic Impact[5]	Estimated Total Regional Economic Impact[6]
PDX	Central Util Plant Bldg & Mech Tunnel	1	$38,000,000	$63,000,000	3	$80,800,000	$242,400,000	$1,560,000,000	$4,680,000,000
	Airfield Runways, Taxiways, Ramps & Lighting	2	$280,000,000	$110,000,000 [8]	3				
	Terminal Conc C and Pass Structure	3	$160,000,000	$543,000,000	3				
	Main Psgr Terminal Bldg	4	$140,000,000	$821,899,000	3				
	New ARFF (Fire) Station	5	$13,000,000	$15,200,000	3				
	P2 Parking Structure	6b	$127,000,000	$196,000,000	3				
	Grd Maint Admin & Shops	13	$6,300,000	$7,800,000	3				
	Grd Maint Facility	14	$2,800,000	$3,400,000	3				
	Grd Maint Facility	15	$200,000	$1,720,000	3				
T6	ICTSI - Berths 604 and 605	7	$23,000,000	$80,000,000 [9]	2	$11,000,000	$22,000,000	$120,000,000	$240,000,000
	Yard trailer maintenance	16	$1,500,000	$1,980,000	3				
	Electric Shop Bldg, SW of Admin Bldg	17	$4,000,000	$1,270,000	3				
	AWC - Berth 601	11	$8,000,000	$35,000,000 [9]	1.5	$2,000,000	$3,000,000	$18,000,000	$27,000,000
T5	Portland Bulk Terminal - Berth 503	8	$20,000,000	$40,000,000 [9]	3	$2,500,000	$7,500,000	$98,000,000	$294,000,000
	Columbia Grain Facility - Berth 501	10	$15,000,000	$25,000,000 [9]	3	$3,000,000	$9,000,000	$180,000,000	$540,000,000
T4	Kinder Morgan - Berth 410-411	9	$13,000,000	$40,000,000 [9]	3	$3,400,000	$10,200,000	$90,000,000	$270,000,000
HIO	Runway 2/20 and 12/30	12	$19,000,000	$66,000,000	1.5	$3,100,000	$4,650,000	$66,000,000	$99,000,000
Total			$870,800,000	$2,051,269,000		$105,800,000	$300,000,000	$2,100,000,000	$6,200,000,000
PORT	HQP2 - Admin offices	6a	$70,000,000	$109,600,000		$105,800,000	$300,000,000	$2,100,000,000	$6,200,000,000

Table 1 Values at Risk – Port of Portland Critical Facilities

3) ImageCat's multi-site software analyzes losses for a large inventory of earthquake simulations, including local earthquakes from sources like the Portland Hills Fault, as well as large events on the more distant Cascadia Subduction Zone. Further information on ImageCat's seismic risk methods is presented in this appendix following this report. .

   For this project, ImageCat modified the SeismiCat Multi-site tool as follows:

   a. ImageCat incorporated specific geologic conditions and ground motion amplification at PDX as developed by GRI / New Albion. These are shown in the figure below.

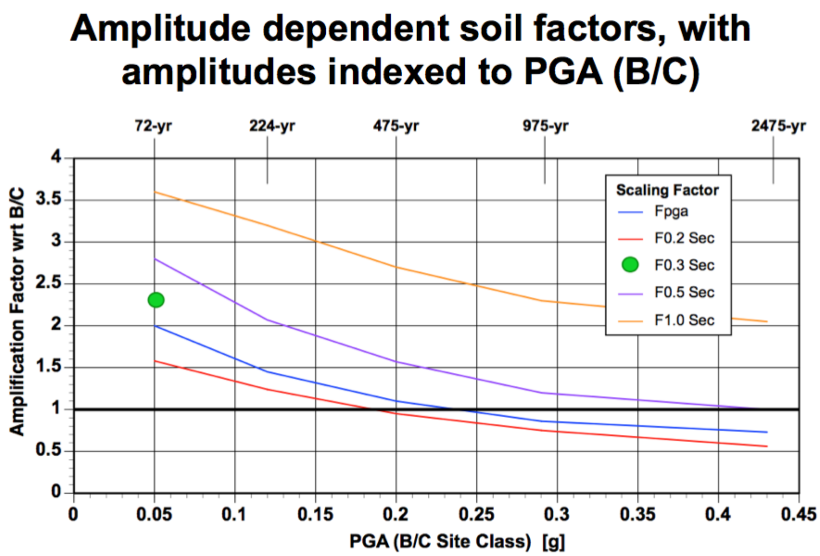


Figure 2 – Site-specific ground motion scaling model for the Port

b. ImageCat implemented PDX-custom logic for analysis of business interruption losses for PDX facilities (buildings and runways). The custom logic was developed in coordination with the Port, with KPFF and HNTB, and is shown below.

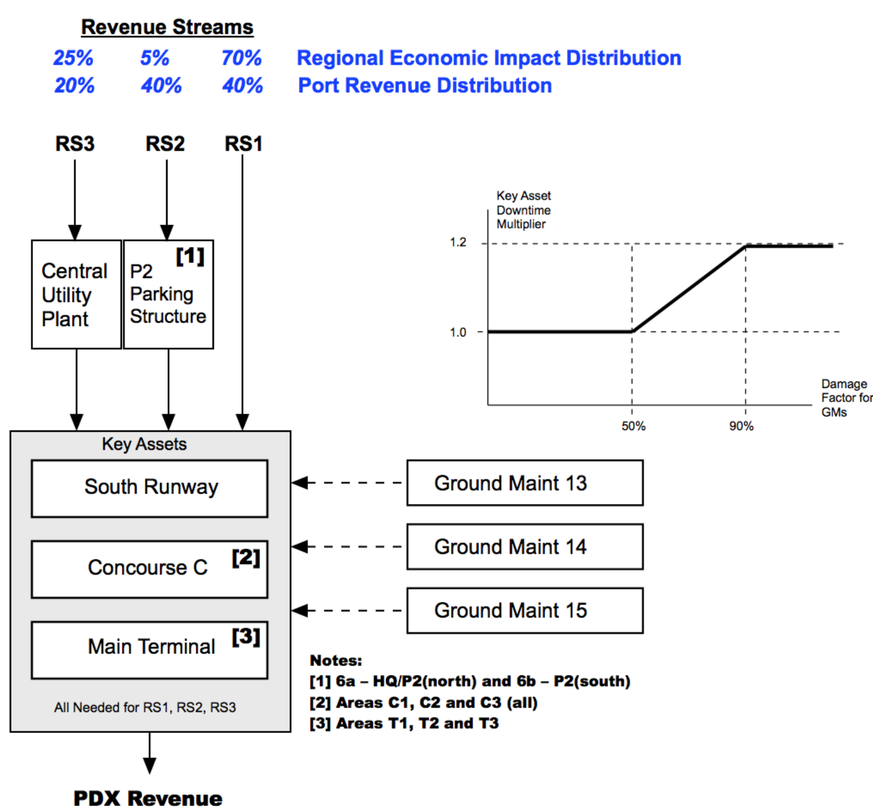


Figure 3 – "Systems" Model for Business Interruption

4) ImageCat worked with GRI / New Albion / BergerABAM for evaluation of risks to marine facilities before and after seismic retrofit and/or replacement, and adapted the risk analysis for simplified benefit/cost analysis.

5) ImageCat conducted risk and benefit/cost analyses for eight different cases, consisting of:

   a. Port Only — Buildings As-Is
   b. Port Only — Buildings, Runways and Marine Facilities As-Is
   c. Port Only — Buildings with Mitigation
   d. Port Only — Buildings, Runways and Marine Facilities with Mitigation
   e. Port and Region — Buildings As-Is
   f. Port and Region — Buildings, Runways and Marine Facilities As-Is
   g. Port and Region — Buildings with Mitigation
   h. Port and Region — Buildings, Runways and Marine Facilities with Mitigation

Status Quo Risks

The figure below shows the results from the seismic risk analysis of the selected critical facilities, under status quo conditions (i.e., prior to any retrofit). The solid lines represent risks to the PDX facilities, including downtime induced by damage to runways. The dotted lines include approximate impacts to the marine facilities. The cases represent a, b, e, and f. The post-retrofit cases (c,d,g,h) are presented at the conclusion of this section.

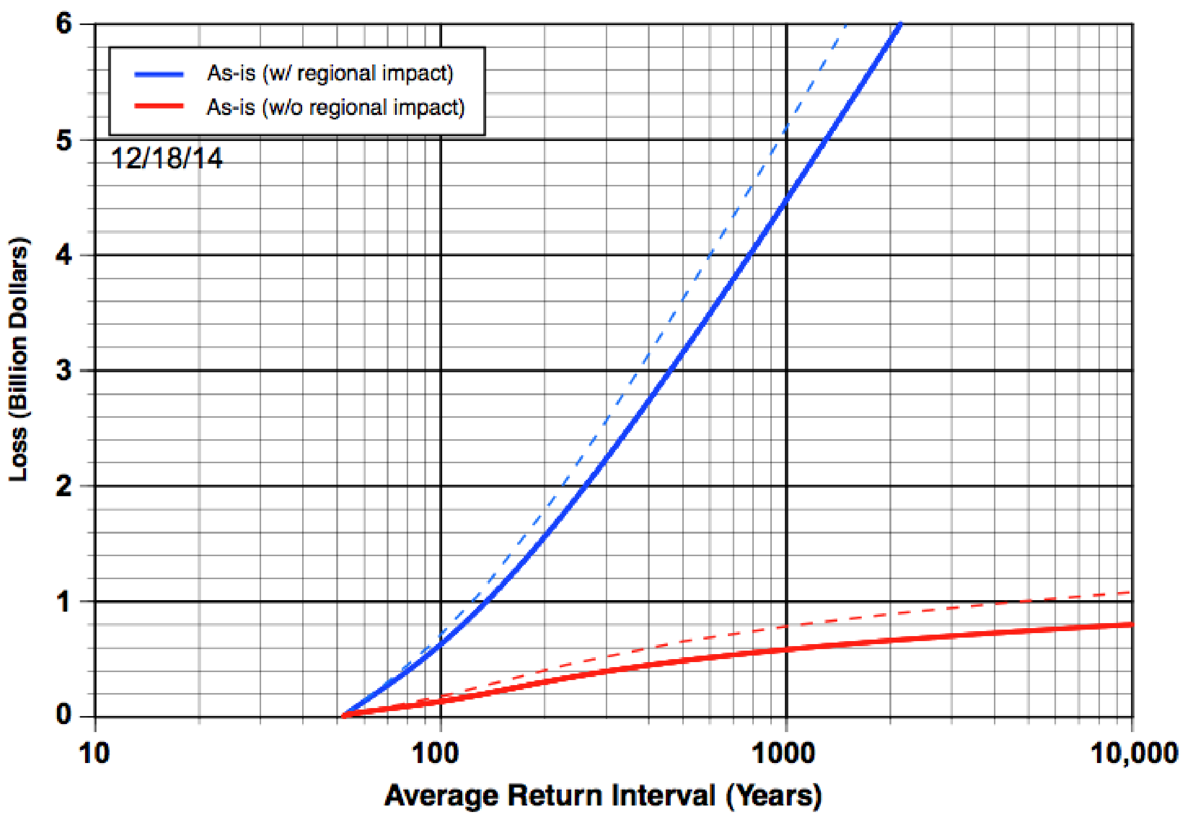


Figure 4 – Seismic Risks to Critical Facilities, As-Is

In considering PDX only (the solid lines) without regional impacts, the 100-year recurrent loss level is on the order of $100M. For increasing return periods and the corresponding increasing in earthquake hazard intensity, losses grow, until PDX losses are about $1B for a return period of 2,000 years. When regional economic impacts are considered, the 100-year recurrent loss level exceeds $600M. The losses to PDX and the region are approach $6B for a return period of 2,000 years.

The increased losses for the blue curve show the dramatic effect of considering the impacts that functionality of PDX airport is expected to have on the region. The dotted lines approximate the additional impact of the marine facilities and runways repair costs, over and above the losses at PDX. In considering actions to take to reduce earthquake losses and improve regional seismic resilience, the blue curves provide strong motivation due to the Port's key role in earthquake recovery for the region.

2 Seismic Risk Methods and Benefit/Cost Analysis

2.1 PDX Buildings

Seismic risks for the PDX Buildings were analyzed using ImageCat's SeismiCat multi-site software. A large set of earthquake simulations is used to represent the full range of future earthquakes, both in magnitude and in location. The set of simulations derives from the models of the 2008 USGS National Seismic Hazard Mapping Project [Petersen et al., USGS Open File Report 2008-1128]. Each simulation depicts the geographic distribution of earthquake hazards for the assumed fault rupture and earthquake magnitude. Losses for each of the buildings are estimated for each earthquake simulation. The damage at each building is computed based on the simulated ground shaking and other seismic hazards (e.g., liquefaction), and the vulnerability of the buildings, as modeled by KPFF. For each earthquake, losses are summed for all of the PDX buildings considered, and the downtime losses are found using the "Systems" Model for Business Interruption (see Section 1). The losses (and their uncertainty) are then related to the probability of occurrence for the simulation, to allow construction of risk curves and other probabilistic results. Further information on ImageCat's SeismiCat multi-site software and risk methods is presented in this appendix following this report. .

2.2 Marine Facilities

ImageCat's SeismiCat multi-site software does not include marine facilities, so those were assessed at specific hazard levels corresponding to defined return periods, in a spreadsheet-based method.

2.3 Benefit/Cost Methods

Benefit / cost analysis (BCA) compares the expected benefits from a candidate retrofit alternatives with the costs to implement the alternative. As such, BCA requires probabilistic risk analysis. Probabilistic risks are defined by severity of consequences and their annual probability (or frequency). Some consequences are easily assigned economic impact (e.g., repair cost). Some consequences require economic analysis (e.g., the financial impacts of critical facility relocation) for the appropriate stakeholders. Other consequences are difficult or controversial to translate into simple economic terms (e.g., death or injury).

The decision framework requires risk analysis for 'status quo' risks for each facility, and risks to each existing facility with the implementation of each retrofit alternative. For calculations of benefit to cost:

- The benefit from each retrofit alternative is found as the reduction in economic (or other) consequence associated with the retrofit alternative with respect to the "status quo" or baseline state.
- The estimated benefit from a particular retrofit alternative for each simulated earthquake event (or hazard level) is multiplied by the annual frequency of the event (or hazard level) to compute the expected annual benefit (i.e., the annual reduction in cost from earthquake damage with implementation of the alternative). The total annual benefit is found by a probabilistic summation of the annual benefit for all earthquake events.
- The present value of future benefits from risk reduction afforded by a particular retrofit alternative is found by assuming that the expected annual benefit occurs each year over the remaining life of the building, and treating this as an annual series of payments. Using time-value-of-money, the present value of this series is computed. This present value benefit is divided by the current estimate of the cost of the retrofit alternative under consideration to obtain a benefit to cost ratio. One important variable is the effective interest rate or the minimum attractive rate of return used in converting an annual series of payments to its present value.

Decisions regarding seismic retrofit alternatives occur within a stakeholder and facilities management context – specific project criteria, goals, decision alternatives and decision frameworks. This broader perspective includes questions such as whether to implement a seismic retrofit alternative beyond minimum code requirements or to accept the level of damage expected for a code-minimum baseline case. In the context of the Port of Portland, region-wide benefits of retrofits at PDX are considered using a "Systems" model for the cost of loss of function of PDX for the region.

Discussion – Benefit / Cost Ratios for Individual Buildings

Benefit-Cost Ratios (BCRs) are strongly affected by how frequently strong ground shaking occurs for the site of interest. For a loss-reduction option to actually reduce earthquake damage, an earthquake has to occur at the site at some point during the remaining useful life of the building after the seismic strengthening has been completed. The amount of payback achieved depends on how often the facility will be shaken and how strongly, as well as on the effectiveness of the seismic damage reduction option in improving life safety and reducing damage and downtime. In areas such as California, damaging strong ground shaking may occur many times during the remaining life of a building considered for seismic improvement. In locations such as Portland, damaging strong ground shaking occur less often, so a building may experience earthquake damage once, or at most a few times.

3 Marine Facilities

3.1 Marine Facility Seismic Performance

		Status quo (As-is) Structure Damage and Downtime				Structure Damage and Downtime with Partial Retrofit						Structure Damage and Downtime for New Replacement Structure			
Berth	Earthquake Return Period	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Partial Retrofit Measure	Cost of Partial Retrofit Measure	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)
410/411	72 Years	410:Minor to moderate repairs; deck repairs, bracing and pile connections; replacement of 10% of piles; 411: deck repairs, pile and pile cap repairs	$7,000,000	6	5 to 8	No feasible partial retrofit alternative to meet 475-year EQ Reconstruction is the mitigation	N/A	(see columns to right)				Minimal damage (est. 5%)	$1,500,000	0	0 to 1
	475 Years	Total reconstruction	$42,100,000	32	26 to 38							Controlled and repairable damage (est. 30%)	$9,100,000	3	2 to 4
	950 Years	Total reconstruction	$42,100,000	32	26 to 38							Reconstruction	$42,100,000	32	26 to 38
501	72 Years	Moderate to significant repairs to structures (est. 50%)	$8,500,000	14	12 to 16	Significant ground improvements around dock and in cells; pile/pilecap strengthening; strengthen conveyor tower supports with new batter piles; strengthen conveyor system tower and bridge connections.	$19,500,000	Minor damage (est. 10%)	$1,700,000	3	2 to 4	Minimal damage (est. 5%)	$900,000	0	0 to 1
	475 Years	Total reconstruction	$27,700,000	28	22 to 34			Significant, repairable damage (est. 50%)	$8,500,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$5,100,000	3	2 to 4
	950 Years	Total reconstruction	$27,700,000	28	22 to 34			Reconstruction	$24,100,000	28	22 to 34	Reconstruction	$27,700,000	28	22 to 34
503	72 Years	Minor to moderate repairs including deck repairs, pile and pile cap repairs; significant trestle reconstruction	$9,000,000	6	5 to 8	Ground improvement program along shoreline, strengthen piles/connections, strengthen concrete beams	$13,100,000	Minor damage (est. 10%)	$3,000,000	3	2 to 4	Minimal damage (est. 5%)	$1,500,000	0	0 to 1
	475 Years	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Significant, repairable damage (est. 50%)	$15,000,000	10	8 to 12	Controlled and repairable damage (est. 30%)	$9,000,000	3	2 to 4
	950 Years	Total reconstruction of a new structure	$37,800,000	32	26 to 38			Reconstruction	$35,750,000	32	26 to 38	Reconstruction	$37,800,000	32	26 to 38
601	72 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21	Ground improvements around the approach trestle and abutment, strengthen trestle piles/connections at batter pile bents, retrofit dolphins	$4,500,000	Minor damage	$1,100,000	1	0 to 2	Minimal damage	$600,000	0	0 to 1
	475 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Significant, repairable damage	$5,400,000	6	4 to 8	Controlled and repairable damage	$3,300,000	3	2 to 4
	950 Years	Total reconstruction of a new trestle and dolphins	$13,300,000	18	15 to 21			Reconstruction of a new trestle and dolphins	$12,700,000	18	15 to 21	Reconstrcution of a new trestle and dolphins	$13,300,000	18	15 to 21
604/605	72 Years	Minor damage, possible rail reconstruction (est. 10%)	$10,000,000	3	1 to 4	Full seismic upgrades described in GeoEngineers report that targets 500-year EQ for repairable damage	$15,300,000	Minor damage (est. 5%)	$5,000,000	1	0 to 2	Minimal damage (est. 5%)	$5,000,000	0	0 to 1
	475 Years	Reconstruction	$100,000,000	30	28 to 32			Significant, repairable damage (est. 15%)	$15,000,000	3	2 to 5	repairable damage (est 10%)	$10,000,000	3	2 to 4
	950 Years	Reconstruction	$100,000,000	30	28 to 32			Reconstruction	$100,000,000	30	28 to 32	Reconstrcution	$100,000,000	30	28 to 32

All Berths Considered	Case	72-years	475-years	950-years
	As-Is	$37.8M	$120.9M	$120.9M
	Partial retrofit**	$12.7M	$71M	$114.7M
	Replacement Structure	$4.5M	$23.2M	$120.9M

** except 410/411

Table 2 – Marine Facility Seismic Performance

The table above presents the results of the evaluation of marine facilities by BergerABAM, New Albion and GRI. For each berth, analyses were performed for seismic hazards corresponding to several return periods – 72 years, 475 years and 950 years. These correspond roughly to an operating basis earthquake level, a design-basis earthquake level, and a rare or extreme earthquake level, respectively. **(See Section 4).** For these various earthquake severities, damage and downtime was predicted, together with expected repair costs. The analyses were then repeated, for the case where the marine facilities were assumed to have completed partial retrofits or replacement with new structures. This then formed the basis for the approximate benefit-to-cost analysis for the marine facilities.

3.2 Benefit/Cost Analysis Results for Marine Facilities

For each hazard level, the reductions in loss afforded by the partial retrofits compared to status quo were multiplied by the annual frequency of occurrence of the hazard level "bin", so that an average annual benefit could be computed. This annual benefit was then converted to present value, and divided by the cost of the retrofit option, to obtain a benefit-to-cost ratio (BCR). The same procedure was followed for the full replacement options. The results are presented in the table below.

Summary, Benefit / Cost Analysis, Marine Facilities

5% discount rate used. Benefits accrued over 50 years.

		Annual Downtime Cost				Benefit / Cost Ratios			
Berth	**Replacement Cost New**	**Port Only**	**Region**	**Partial Retrofit**	**Retrofit Cost**	**Port-Only, Partial Retrofit**	**Port-Only, Replacement**	**Regional Impact, Partial Retrofit**	**Regional Impact, Replacement**
410/411	$42.1M	$3.4M	$90M	No feasible partial retrofit alternative to meet 475-year EQ Reconstruction is the mitigation	N/A	N/A	0.12	N/A	0.77
501	$27.7M	$3M	$180M	Significant ground improvements around dock and in cells; pile/pilecap strengthening; strengthen conveyor tower supports with new batter piles; strengthen conveyor system tower and bridge connections.	$19.5M	0.36	0.18	3.47	3.17
503	$37.8M	$2.5M	$98M	Ground improvement program along shoreline, strengthen piles/connections, strengthen concrete beams	$13.1M	0.29	0.13	1.77	0.92
601	$13.3M	$2M	$18M	Ground improvements around the approach trestle and abutment, strengthen trestle piles/connections at batter pile bents, retrofit dolphins	$4.5M	1.05	0.38	2.85	1.04
604/605	$100M	$11M	$120M	Full seismic upgrades described in GeoEngineers report that targets 500-year EQ for repairable damage	$15.3M	0.51	0.08	2.24	0.38
	$220.9M				$52.4M				

Table 3 – Benefit Cost Results, Marine Facilities

The table shows that all of the partial retrofit options appear to be cost-effective, when regional benefits are included. In particular, partial retrofits to Berths 501 and 601 appear to provide good value. When considering full replacement, again Berths 501 and 601 appear to provide the best return on investment.

4 Runway Performance

The table below presents the results of the seismic evaluation of the north and south runways at PDX by HNTB, BergerABAM, New Albion and GRI. For each runway, analyses were performed for seismic hazards corresponding to several return periods – 72 years, 225 years, 475 years, 950 years and 2,475 years. **(See Section 5).** These levels span the range of earthquake events that may cause damage to the runways. For these various earthquake severities, damage and downtime was predicted, together with expected repair costs. The analyses were then repeated, for the case where the marine facilities were assumed to have completed a program of retrofits. This then formed the basis for the approximate benefit-to-cost analysis for the runways.

		Status quo (As-is) Damage and Downtime				Damage and Downtime with Option #1				
Runway	Earthquake Return Period	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)	Partial Retrofit Measure	Structure Repair Description	Repair Cost Best Estimate	Downtime Best Estimate (months)	Downtime Range (Months)
North	72 Years	minimal damage (< 1% of runway)	$250,000	0.50	0.25 to 0.75	Ground treatment with Stone Columns to depth of 40 ft.	negligable damage (assume 0% of runway)	$ -	0	0
	225 Years	moderate damage (≈ 10% of runway)	$6,300,000	2.0	0.75 to 3.0		minimal damage (< 1% of runway)	$ 100,000	0.5	0.25 to 0.75
	475 Years	Significant repairable damage (≈ 25% of runway)	$15,700,000	6.0	4.0 to 8.0		minimal damage (< 1% of runway)	$ 250,000	0.5	0.25 to 0.75
	950 Years	Reconstruction (≈ 50% of runway damaged)	$31,400,000	8.0	6 to 10		moderate damage (≈ 4% of runway)	$ 2,500,000	1.0	0.5 to 1.5
	2475 Years	Reconstruction (≥ 70% of runway damaged)	$62,000,000	10.0	6 to 12		moderate damage (≈ 10% of runway)	$ 6,200,000	2.0	0.75 to 3.0
South	72 Years	minimal damage (< 1% of runway)	$300,000	0.75	0.5 to 1.0	Ground treatment with Stone Columns to depth of 40 ft.	negligable damage (assume 0% of runway)	$ -	0	0
	225 Years	moderate damage (≈ 10% of runway)	$7,700,000	3.0	2.0 to 4.0		minimal damage (< 1% of runway)	$ 100,000	0.5	0.5 to 0.75
	475 Years	Significant repairable damage (≈ 25% of runway)	$19,300,000	7.0	6.0 to 8.0		minimal damage (< 1% of runway)	$ 300,000	0.75	0.5 to 1.0
	950 Years	Reconstruction (≈ 50% of runway damaged)	$38,500,000	10.0	6 to 12		moderate damage (≈ 4% of runway)	$ 3,100,000	1.5	1.0 to 2.0
	2475 Years	Reconstruction (≥ 70% of runway damaged)	$77,000,000	10.0	6 to 12		moderate damage (≈ 10% of runway)	$ 7,700,000	3.0	2.0 to 4.0

Table 4 – Runway Performance

In particular, the downtime for the runways are critical, as a functioning runway is essential for PDX passenger operations. The cost-effectiveness of seismic improvements to the runways must be considered within the context of the operations of the rest of the PDX facilities. To this end, downtime relationships were developed by HNTB, BergerABAM, New Albion and GRI, as shown in the figure at right.

Separate benefit-to-cost analyses were not run for the runways. Rather the costs and benefits for retrofit of the south runway was included in the benefit-to-cost analysis for the PDX facilities.

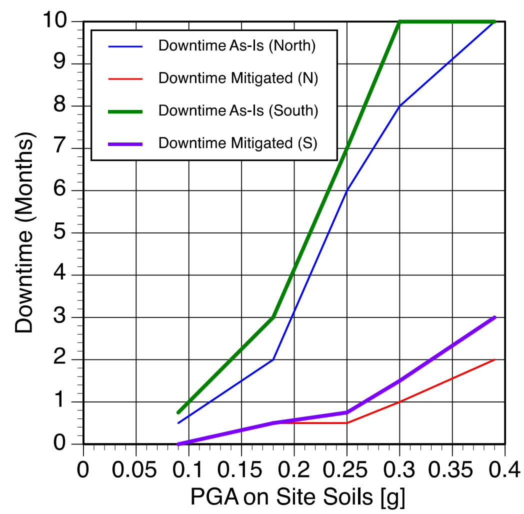


Figure 5 – Runway Downtimes

5 PDX Facility Performance

Section 3 presents details of the structural systems and expected earthquake damage for each of the PDX buildings – the Central Utility Plant (CUP), the terminals and concourses. Section 3 also describes the proposed seismic retrofits for each. KPFF modeled the buildings in their status quo condition in ImageCat's SeismiCat online software, and then modeled the buildings again to simulate their performance after the completion of seismic retrofits. In particular, the effects of soil liquefaction were important, as the buildings are large and the slabs-on-grade will be subject to large liquefaction-induced settlements in high levels of earthquake shaking. For example, the CUP may experience settlements of one foot to 1.5 feet in ground motions with an average recurrence of 500 to 1000 years. High levels of damage may be expected to the slabs on grade, especially in the vicinity of pile-supported columns.

The costs for retrofit for all of the buildings (CUP, terminals and concourses) totaled $200M, so it was of interest to examine and contrast the effectiveness of the retrofits and select the most cost-effective. The figure below shows how effective the proposed retrofits is expected to be for each of the buildings, for the level of seismic hazards that recur on average every 500 years.

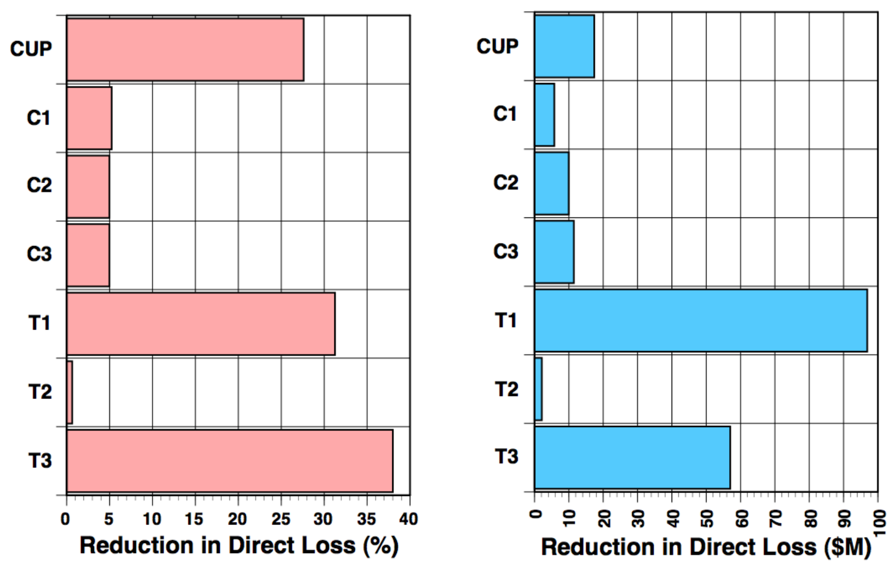


Figure 6 – Comparison of Retrofit Effectiveness

From this comparison, it became apparent that the retrofits for the CUP, T1 and T3 produce the most dramatic benefits. Concourses C1, C2 and C3 already perform well, so the opportunity for damage reduction is not as great for these.

5.1 Seismic Risk Results – PDX Facilities

Figure 7 below presents risk curves for the PDX facilities, showing the impacts of seismic retrofit, with and without regional economic impacts.

The red and blue curves show the status quo risks, with and without regional economic impacts. The difference is dramatic when the business interruption impacts throughout the region are considered. The red curve shows the Port-only perspective for the status quo. For this case, repair costs for earthquake damage are about 2/3 of the values shown by the red curve, with the remainder being Port revenue losses.

The green and the brown curves show the risks projected with a comprehensive program of seismic retrofit for the PDX buildings considered. The green curve includes regional economic impacts, and the brown curve is for Port-only losses. For the post-retrofit case, repair costs for earthquake damage are about 9/10 of the values shown by the brown curve, with the remainder being fairly small Port revenue losses.

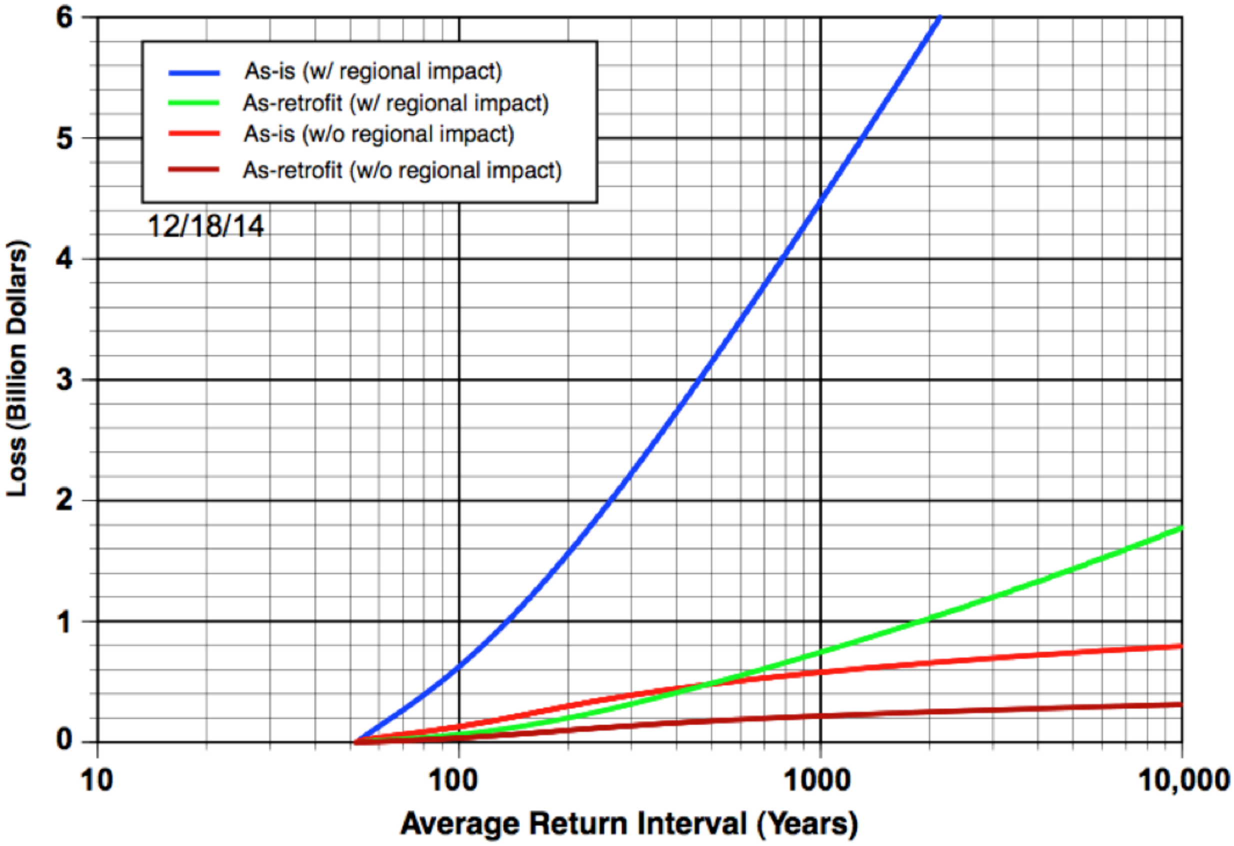


Figure 7 – Building Damage + PDX B.I. (including runway impacts)

5.2 Benefit/Cost Analysis Results

Table 5 shows the results of benefit / cost analysis for the complete set of PDX assets, with and without regional impacts.

Average Annual Loss (AAL) — **Benefit**

All PDX Retrofits + South Runways

As-is		As-retrofit		∂AAL		n	rate	Present Value	
With regional	w/o regional	With regional	w/o regional	With regional	w/o regional			With regional	w/o regional
24,365,730	3,927,906	3,660,681	1,372,155	20,705,049	2,555,751	50	0.05	377,989,831	46,657,600

	With regional	w/o regional
South Runway Damage Reduction	$ 2,644,571	$ 2,644,571
Total	380,634,402	49,302,171
BCR	**1.4**	**0.18**

CUP	$ 16,000,000
C1	$ 14,000,000
C2	$ 31,000,000
C3	$ 36,000,000
T1	$ 47,000,000
T2	$ 36,000,000
T3	$ 20,000,000
South Runway	$ 67,000,000
	$ 267,000,000

Table 5 – Benefit / Cost Analysis Results, Comprehensive Retrofits

Table 6 shows the improved results for the reduced set of assets (CUP, T1 and T3 and the South Runway), with and without regional impacts. This selected set of assets allows PDX operations to resume, preserving the revenue, and so optimizing benefits.

Alternative with Selected Retrofits (CUP, T1, T3 and South Runway)

As-is		As-retrofit		∂AAL		n	rate	Present Value	
With regional	w/o regional	With regional	w/o regional	With regional	w/o regional			With regional	w/o regional
24,365,730	3,927,906	6,175,717	1,716,138	18,190,013	2,211,768	50	0.05	332,075,521	40,377,872

	With regional	w/o regional
South Runway Damage Reduction	$ 2,644,571	$ 2,644,571
Total	$ 334,720,092	$ 43,022,443
BCR	**2.2**	**0.3**

Selected Mitigation Costs

CUP	$ 16,000,000
T1	$ 47,000,000
T3	$ 20,000,000
South Runway	$ 67,000,000
	$ 150,000,000

Table 6 – Benefit / Cost Analysis Results, Selected Retrofits

5.3 Limitations

The economic contributions from life-safety enhancements are not considered in the model. Benefit cost analysis under FEMA procedures would allow such benefits to be considered, but only with a considerable analysis effort. As the terminals, concourses and offices are the principal high-occupant areas, life-safety benefits would have some effect, increasing BCRs for the case when regional economic impacts are not considered. For the case where regional economic impacts are included, the regional impacts are so much greater in magnitude that the change in BCRs would be negligible.

The dependency of building functions and operations on lifelines (roads, railroads, waterways) and utilities (power, gas, water, telecomm) was also not considered. With liquefaction and other seismic hazards, damage to lifelines and utilities may impact downtimes and increase regional economic losses.

ImageCat's Seismic Risk Methods

1. Overview

Analysis methods for probabilistic portfolio seismic risk assessment first emerged in the late 1980s and early 1990's [14, 19, 20, 21] based in part on previous development in lifeline earthquake risk analysis [18]. Since that time, these approaches to portfolio seismic risk have been adopted, extended and standardized by catastrophe modelers for the property insurance industry. ImageCat's models are developed and maintained by some of the original contributors.

Probabilistic portfolio seismic risk assessment relates a return period (or probability) with the portfolio-wide loss level (aggregate loss). This is done considering all foreseeable future seismic events, rather than for some arbitrary subset of maximal events. By considering all damaging earthquakes, a comprehensive and balanced picture of risk emerges.

ImageCat's portfolio seismic risk tool uses a comprehensive set of individual earthquake simulations, called an 'event set.' Each simulation has a geographic distribution of ground shaking calculated from empirical attenuation relationships (e.g., PEER's NGA relationships), with adjustment for local site conditions. By computing portfolio-wide losses for each simulation, we directly account for the site-to-site correlation of loss within each earthquake event.

ImageCat's event set follows the methods deployed in the USGS National Seismic Hazard Mapping Project [15]. The same earthquake source (fault) modeling and attenuation relationships are used, and the USGS methods are adapted to produce discrete earthquake events. ImageCat's event set systematically simulates earthquakes on known faults as modeled by the USGS, in each possible fault rupture location, over the full range of magnitudes causing damage, and including background seismicity. Each event simulation provides the spatial distribution of shaking and other hazards, and each event is assigned an annual frequency of occurrence consistent with the USGS source model. Hazard uncertainties relating to simulations, such as maximum magnitude, fault rupture area versus magnitude, attenuation uncertainties, etc., are carefully accounted for in event set construction and usage. Each loss at each site, and the resultant portfolio-wide loss is computed, with its full statistical distribution. Combining losses must follow sound statistical practice to meet actuarial standards.

The vulnerability of buildings, equipment and contents follow methods developed by the authors [11] called Code-Oriented Damage Assessment (CODA), as well as ATC-13. Probabilistic models based on HAZUS®MH technology are also available, producing "expected loss" results rather than a full statistical distribution. The vulnerability models relate earthquake damage repair costs to earthquake ground shaking intensity as measured by Spectral Acceleration, Sa. For CODA and ATC-13, the variability of damage for a defined hazard state is modeled as a function of the quality of the data, based on the level of engineering investigation [1, 3, 11]. For HAZUS, ImageCat's models follow Beta values established by NIBS (i.e., by Kircher) for HAZUS AEBM, accounting for total shaking hazard uncertainty, uncertainty in Capacity Spectrum solution point, and uncertainty in fragility medians for structural and nonstructural components.

Portfolio losses are displayed in risk curves that plot portfolio-wide loss levels as a function of the average return period for the loss. The loss curves present the full range of financial risks, rather than just a single scenario point estimate, and may be examined at any return period of interest. For CODA and ATC-13 models, loss severity is typically presented as portfolio-wide "Probable Loss," found by combining loss uncertainty with event frequencies in constructing the curve. Statistical models are used to allocate the losses to the various stakeholders, and risk curves may be presented from any of these viewpoints – owner, lender, insurer, etc.

ImageCat provides various outputs and de-aggregations of risk to identify key risk contributors, and enable intelligent risk mitigation:

- maximum loss modeled and its associated scenario
- 475-year Scenario Expected Loss by building
- Average Annual Loss by building
- Geographic Correlation Index [10], identifying buildings by catastrophic portfolio impact
- risk curves segregating regional exposures
  – e.g., curves for northern California, southern California and all California
- risk curves with and without liquefaction-induced losses
- risk curves with and without "demand surge"
- risk curves with and without separate treatment of inter-event and intra-event uncertainty
- risk curves comparing loss contribution from building repairs, damage to contents and losses from downtime

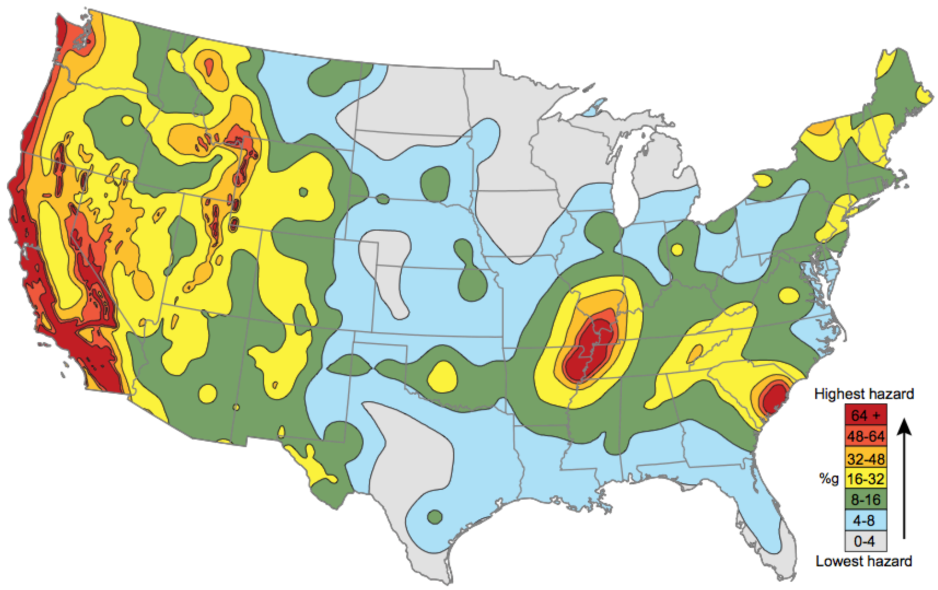
ImageCat's seismic risk models for insurance typically consider damage to the building and its contents from ground shaking and liquefaction-induced foundation failures on flat sites. ImageCat's models exclude tsunami damage, damage from surface fault rupture through building foundations, earthquake-induced fires, and damage from water leaks in plumbing and fire sprinkler piping. ImageCat's seismic risk models for insurance may also consider business interruption related to downtime from building vacancy needed for repair following earthquakes. ImageCat's business interruption models typically do not consider downtime due to off-site factors such as may result from damage to utilities (water, sewer, power), loss of data or other communications, or impairment of site access. Special studies may consider one or more of these excluded effects.

2. Seismic Hazards

"Event Sets"

Geographic correlation of damage and loss is of primary concern in the seismic risk assessment of a geographically distributed portfolio, so the physical size of the source fault rupture must be properly modeled, and the spatial distribution of shaking modeled with appropriate ground motion attenuation relationships.

ImageCat's event set follows the methods deployed in the USGS National Seismic Hazard Mapping Project [15]. The USGS model has been periodically updated (1996, 2002, 2008) and serves as the authoritative national basis for seismic building codes and national risk models (e.g. HAZUS). The same earthquake source (fault) modeling and attenuation relationships are used, and the USGS methods are adapted to produce discrete earthquake events.



Ground Shaking Map from USGS

Each event is associated with an annual frequency of occurrence (number of events per year, typically << 1), where the annual frequencies are derived from fault activity, magnitude and fault rupture location "sampling." The 'event set' systematically exercises the full range of earthquake magnitudes and rupture locations for each seismic sources, including known faults and background seismicity. The set of scenarios and event frequencies is carefully constructed so that the ensemble accurately reproduces the severity and frequency of ground shaking for the region of interest, as modeled by the USGS. These simulations usually involve many thousands of scenarios in each complex tectonic region such as southern California, where numerous known and unknown faults exist.

Ground Shaking Uncertainty

Empirical ground motion prediction relationships (e.g., PEER's NGA relationships, Earthquake Spectra -- February 2008, Volume 24) are subject to uncertainty, modeled as a lognormal standard error.

The uncertainty in predicting ground motion amplitudes from the future earthquakes is one of the major sources of uncertainty with significant impact on earthquake risk analysis. The random variability in ground motion prediction from attenuation relations can be partitioned into two parts: the inter-event term (σ) and intra-event term (τ) (Joyner and Boore, 1981; Abrahamson and Youngs, 1992; Abrahamson and Youngs, 1997). The inter-event term accounts for the discrepancy in mean ground motions recorded from earthquake to earthquake. For instance, an earthquake that has a higher than average stress drop is expected to generate ground motions systematically higher than average. The intra-event term, on the other hand, measures the randomness of ground motions across a geographic region from a single earthquake. The two uncertainty terms are typically treated as independent variables, and the total variance at a single site is the combination of the two terms $(\tau^2 + \sigma^2)$.

For the risk analysis of a geographically distributed system, the implications of the two terms are distinct and important. For a uniformly distributed portfolio that consists of a large number of independent structures, the influence of the intra-event variation to portfolio risk tends to be reduced as portfolio size and number of structures increases. In the limit, the predicted portfolio loss would approach the expected loss based on the average ground motion from an earthquake. However, the inter-event variation has a large effect on portfolio-wide aggregate loss because it produces systematically higher or lower ground motions occurring at all sites during the same earthquake. For small-to-moderate portfolios with spatial clustering and properties varying by dollar exposure and vulnerability, both terms can be significant to portfolio risk analysis.

ImageCat segregates the ground shaking prediction uncertainty into these inter-event and intra-event terms and accounts for the risk impacts separately.

Site-Specific Hazards

Site ground conditions affect the intensity and duration of ground shaking, as well as the shape of the ground motion response spectrum. In comparison to rock sites, soft soils amplify moderate ground motions, extending the duration of ground shaking, and shifting seismic energy to longer periods. At very high levels of shaking, soft soils may actually reduce peak ground motions, compared to rock. In the ImageCat event set, ground shaking is computed for a standard ground condition ("Site Class") and adjusted for actual site conditions as determined from regional maps [e.g., 24] using methods consistent with building codes and national standards (IBC, NEHRP, etc.). Where detailed site-specific information is obtained, as from a geotechnical investigation report, the actual ground condition is input and used rather than a mapped condition.

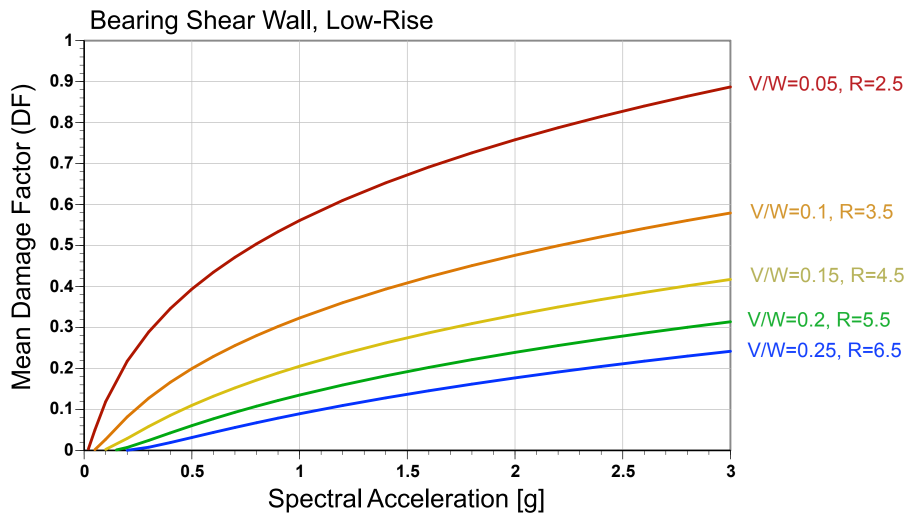
Liquefaction Susceptibility

ImageCat uses proprietary damage algorithms for damage from liquefaction-induced settlements on flatland sites. We consider damage to slabs-on-grade and building structural and nonstructural systems from settlement and uplift from effects of liquefaction at the ground surface, but we exclude damage from lateral spreading, lurching, etc. ImageCat's models consider earthquake magnitude, shaking intensity and liquefaction susceptibility at the site, and the building foundation system.

3. Seismic Vulnerability

CODA – ImageCat adapted the published CODA model [11] for use in probabilistic seismic risk modeling. CODA building structural classes include:

- Wood Frame
- Reinforced concrete or masonry shear wall
- Reinforced concrete or steel moment frame
- Concentric steel braced frame
- Eccentric steel braced frame
- Unreinforced masonry, bearing wall (retrofit and non-retrofit)
- Unreinforced masonry infill frame (concrete frame and steel frame)
- Steel light frame
- Mobile homes and prefabricated housing (anchored and unanchored)
- Precast concrete tilt-up
- Precast concrete other than tilt-up
- Dual systems (moment frames + steel braced frames, or concrete or masonry shear walls)



CODA Damage Curves for Low-rise Shear Wall Buildings

All of these systems exist in low-rise construction. Most also exist in mid-rise and high-rise construction. In addition to height and building structural class, CODA parameters for building seismic risk assessment include:

Fundamental structural period (T)
Design strength (V/W or Cs, in LRFD units)
Response modification factor ("R" – similar to ASCE 7 or UBC 1997 R-factors)

In CODA, shaking-induced damage is a function of a demand-to-capacity ratio (DCR), where the demand is the 5% damped spectral acceleration at the building's fundamental period, and capacity is the product of design strength (Cs, or V/W, at LRFD level) and the Response Modification Factor, R.

$$\text{Mean Damage Factor } DF = f(DCR)$$
$$DCR = Sa(T) / [Cs \times R]$$

Because the CODA models utilize these parameters from seismic building codes, CODA models are easily adapted to year of construction and location. The evolution of building code through time and by location (seismic zones) is straightforward to trace, so the CODA models can make a good initial estimate of seismic resistance if the structural type is known. With engineering investigation, the specific features of the particular building in question are easily accommodated, using the same engineering parameters found in building codes.

ATC-13

ATC-13 [16] was intended to estimate earthquake damage to buildings in coastal California (i.e., UBC Seismic Zone 4). ImageCat's tools provide damage algorithms adapted from ATC-13 for damage from earthquake shaking. The ImageCat implementation of ATC-13 takes advantage of the development of CODA methods [11] to adapt and extend the coverage of ATC-13 to other zones and to older construction predating UBC Seismic Zone 4 (i.e., prior to 1976), and to account for the magnitude dependence of spectral shapes.

Damage Uncertainty

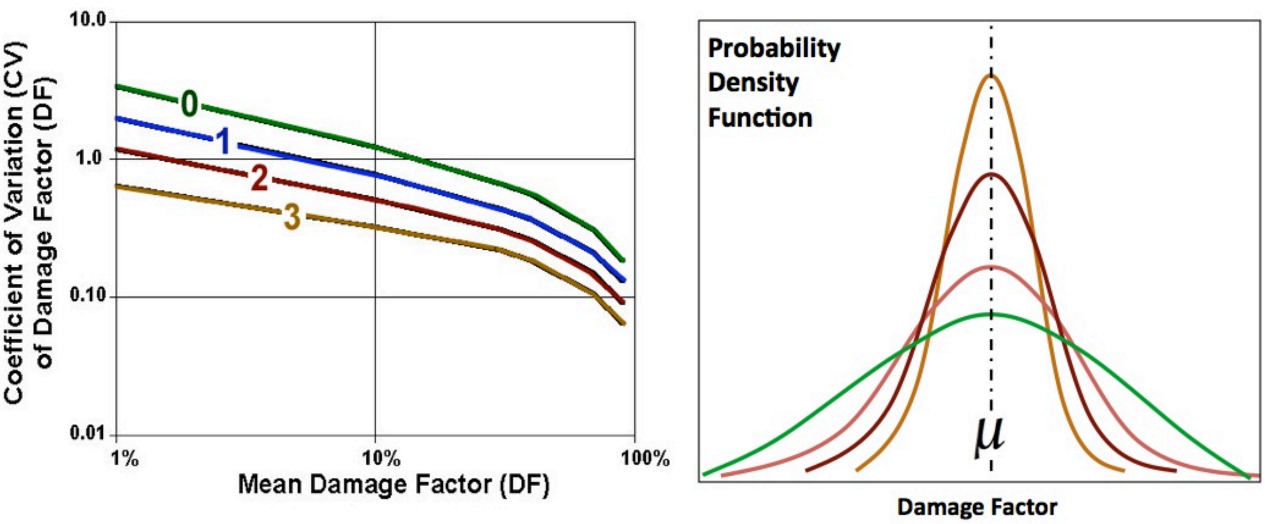
Models for building damage from earthquake ground shaking must address not only the expected value of damage, but also the variability or uncertainty of damage, given a particular hazard state. ImageCat uses a statistical distribution for Damage Factor (DF), given the hazard state as defined by spectral acceleration (SA). This is a *conditional* distribution, since the hazard state is “known” (i.e., presumed or measured with precision). In risk computations, we account for the variability of ground motions separately from building damage variability. For the hazard state, we use a lognormal standard error associated with the attenuation relationships.

Better data can improve the accuracy and precision of damage estimates. The amount of uncertainty can be reduced by engineering investigation to confirm the building class, observe quality and configuration, and more precisely estimate the structural parameters (T, Cs, and R). To construct the uncertainty relationship for Damage Factor for CODA as a function of Level of Investigation, we used the CODA model itself, and perturbed the input variables within any given framing system to obtain a distribution of resulting damage. Specifically, we constructed statistical distributions of the uncertainties associated with capacity (V/W), ductility (R) and building period (T) specific to each Level of Investigation and use a Monte Carlo simulation to obtain a distribution of DF. The resulting damage distribution represents the "reducible uncertainty" which varies with the level of engineering investigation.

ImageCat identifies four “Levels of Investigation”:

0 – Desktop study (location, age, height and occupancy known)
1 – Visual walk-through survey conducted by a Professional Engineer (P.E.)
2 – Visual walk-through + design drawing review conducted by a P.E.
3 – Visual walk-through + design drawing + detailed engineering analysis by a P.E.

The figure below shows the ImageCat uncertainty model for building damage (see also [11]).



CODA Damage Uncertainty as a Function of Level of Investigation

Note that substantial uncertainty remains even after engineering investigation. The perturbation of parameters within the model does not account for the variation in damage that remains when all of these parameters are known to within narrow bounds. This variation in damage results from “what is not explained by the model,” often referred to as random or aleatory uncertainty. The sources of such variability are many. For example, the CODA model treats a building as an elastic single-degree-of-freedom (SDOF) oscillator, and does not explicitly account for structural irregularity. Real buildings are much more complex structures, responding with many degrees of freedom. Structural damage introduces nonlinear and hysteretic behaviors that directly violate the elastic SDOF assumption. Real structures often defy even advanced structural models. ImageCat’s model assumes that the aleatory or "irreducible" uncertainty is of the same order as the "reducible uncertainty" and is independent of the factors considered in the perturbation of parameters.

4. Portfolio Seismic Risks

Risk computation process

For each earthquake simulation, the ground shaking is found at each site and the seismic vulnerability relationship for the property is used to estimate the loss rate (repair cost as a fraction of replacement value). The repair cost in dollars is then computed by multiplying the loss rate by the buildings’ replacement value. Losses are added from site-to-site to obtain the aggregate loss or portfolio-wide loss. Uncertainties in ground shaking intensity and in vulnerability are tracked through the loss computation process, and stakeholder models are used to allocate losses to the various parties (i.e., the owner, lender or insurer) in a statistically sound fashion.

Risk Curves

The results of probabilistic risk analysis are often presented in the form of “risk curves,” plotting the severity of risk consequence (e.g., dollars or downtime) versus annual probability of exceedance. After assessing consequences for an event set, such a plot may be constructed by proceeding from the highest consequence to the lowest, and accumulating event annual frequency of occurrence to find frequency (and the related probability) of exceedance.

ImageCat plots risk curves with the vertical or Y-axis depicting the severity of risk consequence (e.g., dollar losses or downtime), and the horizontal or X-axis as average return period, in years. In this way, points on the risk curve are easily identified as the “100-year loss” or the “475-year loss.” The average return period is found as the multiplicative inverse of the annual frequency of exceedance:

$$T = 1/f_e$$

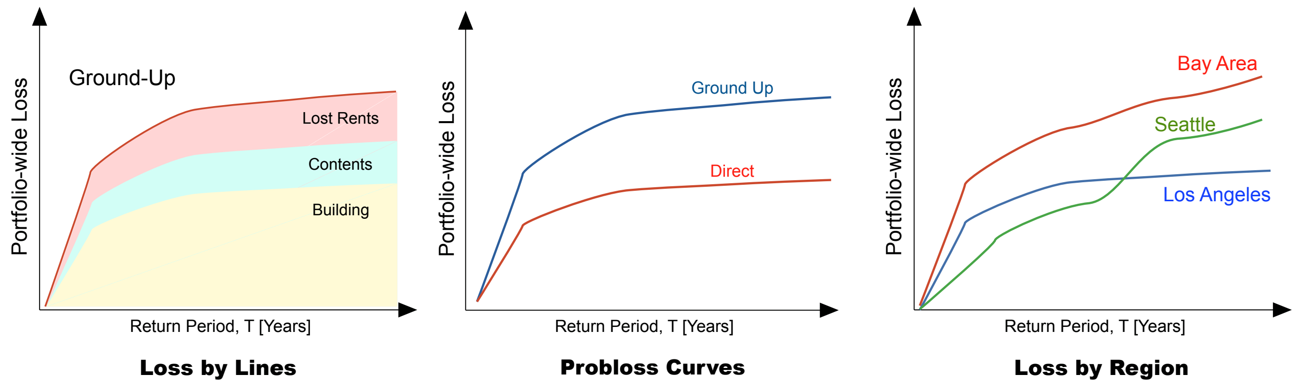
Where T = average return period [years]

f_e = annual frequency of exceedance

An ‘Expected Risk Curve’ is a risk curve found from the mean loss for each event and the associated event annual frequency of occurrence, using the accumulation process described above. Using the statistical distribution of losses for each earthquake, we can also construct curves showing curves from the 10th centile and 90th centile nonexceedance values.’ Probable Loss’ is found by re-binning the event loss distributions so that loss is related directly to probability.

Risk curves may be constructed with loss by “Line of Coverage,” so the relative contribution of building damage, contents damage and business interruption losses can be examined.

Risk curves may also be constructed to overlay and compare loss in independent seismic regions. Exposure is grouped by seismically-independent regions, such as northern California, southern California and the Pacific Northwest, with no (credible) seismic events affecting more than one region. In this way, the degree of seismic diversification may be assessed as a function of return period.



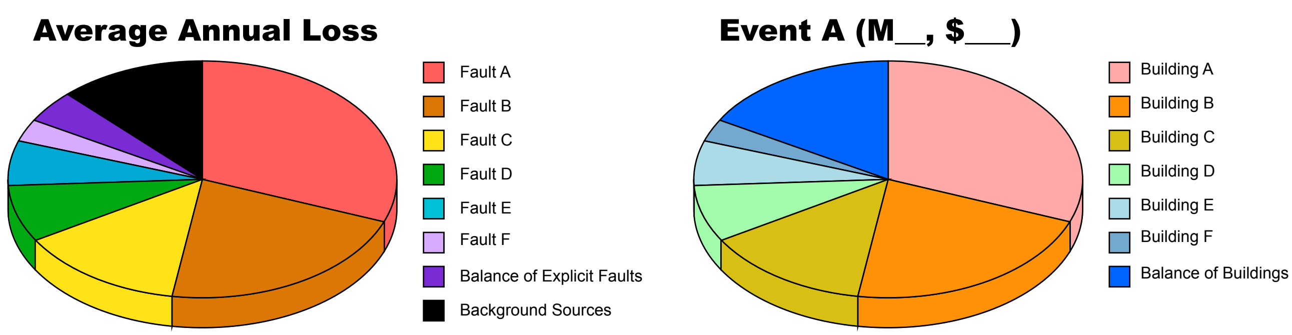
Risk Curves

Single-site Risk Outputs

Single-site risk outputs may include Scenario Expected Loss (SEL) and Average Annual Loss (AAL). See the Glossary for further definition.

Other Outputs

Other outputs may include "contribution plots" – pie charts to de-aggregate Average Annual Loss or portfolio-wide loss for a particular scenario. The de-aggregations may be according to seismic source or by property. These plots are used to identify the sites or sources driving portfolio risks.



Contribution Plots

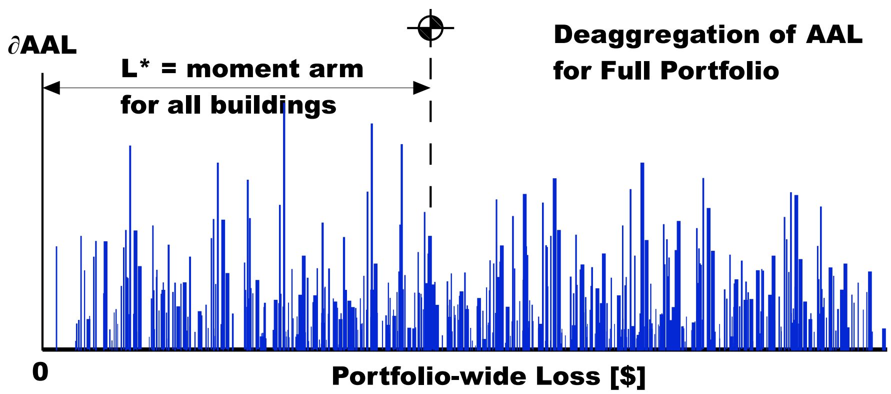
Geographic Correlation Index (GCI)

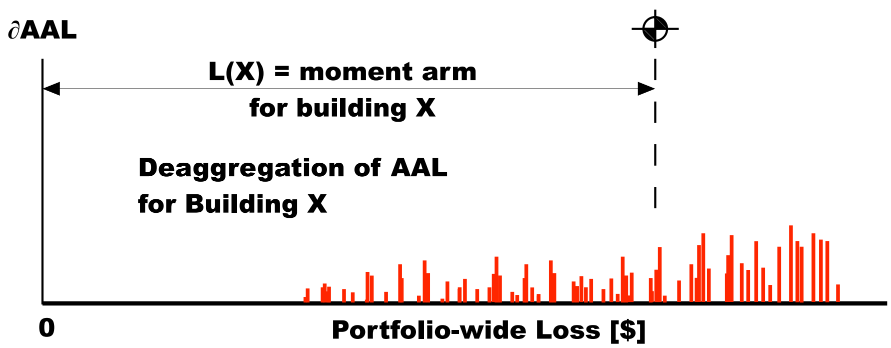
The Geographic Correlation Index (GCI), modified from [10], is a measure of how correlated a particular building's losses are with the high-consequence losses for the portfolio. GCI = 1.0 for an "average" building in the portfolio. A GCI greater than 1.0 indicates that the building in question participates more strongly in high-severity portfolio-wide losses. The building's losses are correlated with the concentrations of property exposure that drive catastrophic risk. A GCI less than 1.0 indicates that the building in question participates mostly to lower-severity portfolio-wide losses, and so contributed to geographic diversification within the portfolio.

Geographic Correlation Index is an index derived from a de-aggregation of Average Annual Loss (AAL). In simple form, AAL = ∑ Loss(i) x f(i) for all events (i) in an "event set", where each event has annual frequency f(i). It may be calculated for a single building, or for the overall portfolio. Each building's incremental contributions to AAL (∂AAL) can be tracked separately, earthquake by earthquake.

$$\partial AAL(j,i) = Loss(j,i) \times f(i) \quad \text{for building (j) in events (i)}$$
$$\text{For building (j), } AAL(j) = \sum \partial AAL(j,i) \text{ summed for all events, i}$$

Each event (i) produces a portfolio-wide loss with severity S(i). By distributing a particular building's incremental AAL contributions according to portfolio loss severity, it becomes clear whether the building contributes only to low-level portfolio-wide losses, or whether the building is a key contributor to severe portfolio-wide losses. GCI is defined as the moment arm to the "center of gravity" of a building's ∂AAL distribution, divided by the moment arm of the total portfolio ∂AAL distribution. This is illustrated below.





GCI(i) = L(i) / L*

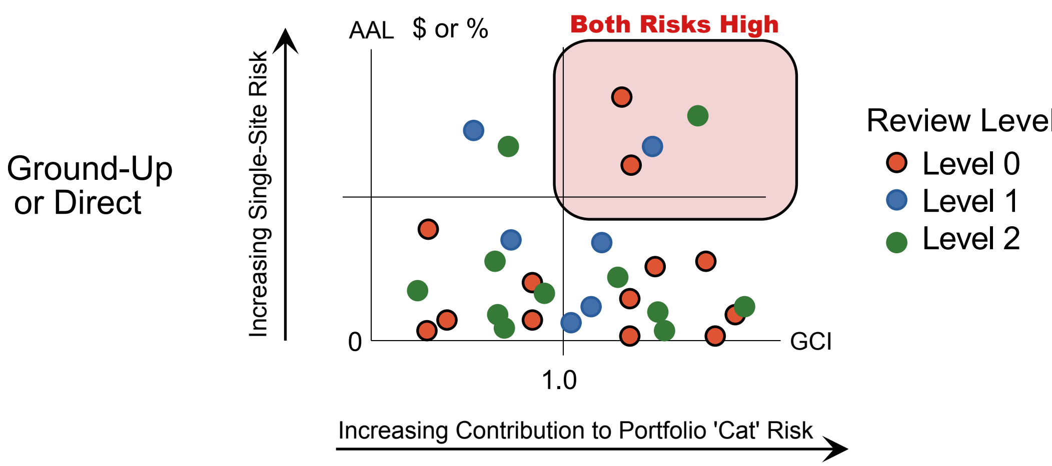
A GCI greater than 1.0 indicates that the building in question participates more strongly in high-severity portfolio-wide losses. The building's losses are correlated with the concentrations of property exposure that drive catastrophic risk. A GCI less than 1.0 indicates that the building in question participates mostly to lower-severity portfolio-wide losses, and so contributed to geographic diversification within the portfolio.

Risk Map

A "Risk Map" plots single-site risk versus portfolio loss correlation, to identify those sites having high risks on an individual basis and high contribution to severe portfolio-wide risks. These buildings are the "drivers" of portfolio seismic risks, and are candidates to investigate for the most effective improvement of modeling data, and for potential loss-reduction measures.

The Y-axis is Average Annual Loss, a single-site risk parameter presenting the long-term earthquake loss per year. It may be plotted in dollars/year or as an annual damage factor (repair cost/replacement value). The X-axis is Geographic Correlation Index (GCI).

The Risk Map plots all buildings into this X-Y space, and the upper right quadrant has buildings with higher-than-average AAL and GCI >1.0. These are the risk drivers. The symbols for each building are color coded to indicate its current level of investigation (Level 0 = Desktop Study with high uncertainty; Level 3 = Detailed Engineering Study with low uncertainty)



Risk Maps

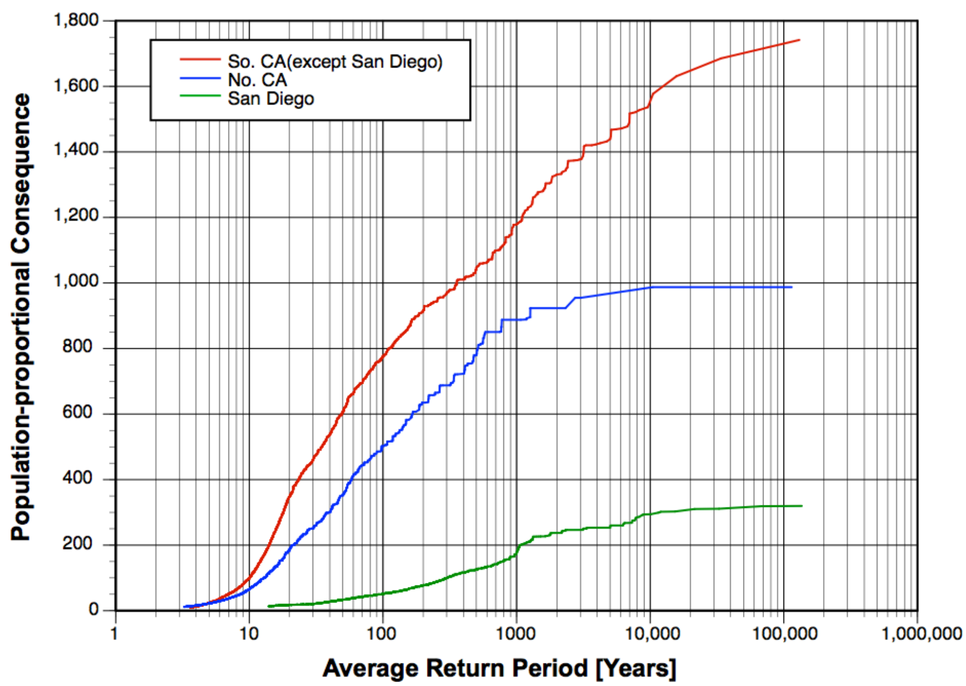
Demand Surge

In small-to-moderate sized earthquakes within prepared communities, the resources to respond, repair and recover are available locally. The costs and time to repair and restore earthquake damage to property and to resume business are not greatly different from the costs and schedules for normal construction. Most earthquake risk models start with these costs and timeframes as a baseline, and most earthquake experience and damage statistics are drawn from moderate earthquakes.

In large-to-great earthquakes within prepared communities (and in smaller earthquakes where preparation lacks), the local resources to respond, repair and recover are become overwhelmed. The costs and time to repair and restore property and resume business increase with respect to the costs and schedules for normal construction. This effect is known as *demand surge*.

ImageCat developed a simple model to provide approximate demand surge effects, in which we assume that the damageable properties and infrastructure elements are distributed geographically in proportion to local population [23]. In other words, residential population distribution is used as a proxy for a global exposure database (GED). This is approximate, since in urban areas we have not only the residential construction as reflected in population numbers drawn from the census, but also separate concentrations of businesses (e.g., offices, mercantile, warehouses, manufacturing, etc.), government, education and infrastructure elements that are distributed differently than the (residential) population. We model the vulnerability of residential construction – essential for workers to remain in place to serve the community. This simple model of exposure (number of people), vulnerability (typical wood framed residence), and hazard (ground shaking based on USGS National Hazard Mapping Project and liquefaction susceptibility based on regional maps) provides the basis for an approximate representation of damage density and an index for the magnitude of regional damage. From this damage picture, inferences are drawn and a heuristic model of the increased damage and downtime is constructed.

ImageCat's preliminary model for demand surge associates a multiplicative factor to inflate the repair cost and the projected downtime with specific causative large-to-great earthquakes, based



Expected Consequences in California

on the global effects for these particular earthquakes, as found by the indices of aggregate consequence described above. For specific client portfolios analyzed for insurance or other purposes, these factors are used to inflate the losses and downtimes at each site, since these earthquakes are shown to produce large regional impacts, competing with the affected properties in the specific portfolio for the resources needed to repair and restore them. Downtimes are simply scaled up. Repair costs are increased by scaling up replacement values to account for the added cost of construction in the aftermath of the earthquake. The scaling factors are regional. In California, the highest values in the Los Angeles metropolitan area. Moderate values are used the San Francisco Bay area, and lower values in the San Diego region. Demand surge modeling was also done for the Pacific Northwest and the New Madrid Seismic Zone. The demand surge factors assigned consider past regional studies [4, 5, 12, 13, 17, 22], local post-earthquake demands and the capacity to respond in the affected regional and surrounding areas.

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Appendix 6 – Dependency of the Port of Portland on Regional Lifelines and Utilities

(ImageCat, Inc.)

Appendix 6 – Dependency of the Port of Portland on Regional Lifelines and Utilities

C. K. Huyck, ImageCat 2/2/2015

Introduction

This chapter provides detailed maps of the facilities analyzed throughout the Port of Portland and related airport structures. Additional maps are included to illustrate the regional exposure to hazardous materials, liquefaction hazards, regional ground motion, and the dependency upon lifeline networks in the area. This section does not provide an analysis of lifeline impacts, but does suggest options for assessing lifeline risk and remediation opportunities.

Locations of Top Priority Facilities

The project team worked with the Port of Portland to identify the top 18 facilities of interest. The final ranking is available in Table 1, and Figure 1 provides a map of these facilities, with the priority of the top priority facilities in red lettering. The top priority sites were critical facilities at Portland International Airport (PDX), followed by key marine terminal berths, the Hillsboro Airport main runway, PDX maintenance facilities, and the marine relief and economic recovery activities at Terminal 6.

The priority facilities addressed in this study represent five primary locations: Portland International Airport, Terminal 6, Terminal 5, Terminal 4, and the Hillsboro Airport. The priority facilities are mapped in yellow in Figure 1, and additional properties owned by the Port but not associated with priority properties are mapped in blue. Figures 2-9 provide detailed maps of the facilities. Detailed description of the selected priority facilities can be found in the body of the report. Outside of the mapped extent in Figure 1, there are several properties owned by the Port of Portland including: Government Island, McQuire Island, Troutdale Airport, and office facilities at Gresham Vista Business Park and Troutdale Reynolds Industrial Park (of these facilities, all but Gresham Vista Business Park are exposed to a high potential of liquefaction).

Table 1: Port of Portland Priority Facilities

Final Rank	Location	Street Address	Location Description	Brief description of reasoning for placement and ranking on the list
1	PDX	7110 NE Airport Way	Central Util Plant Bldg & Mech Tunnel	Provides heating, cooling, hot water, power, and back-up power to the terminal
2	PDX	either N or S (need one)	Airfield Runways, Taxiways, Ramps & Lighting	Vital to both emergency/relief response and economic recovery
3	PDX	7000 NE Airport Way	Terminal Conc C and Pass Structure	CCC houses the Comm center and Emergency Operations Center (EOC) (C3 on diagram) - coordination of emergency response
4	PDX	7000 NE Airport Way	Main Psgr Terminal Bldg	First priority for the terminal building is the South node, (T2 on diagram) which contains the Port Police station, first responders
5	PDX	5250 NE Marine Dr	New ARFF (Fire) Station	First responders
6a	PDX	7200 NE Airport Way	Admin offices	Port headquarters, vital to Port economic recovery.
6b	PDX	7200 NE Airport Way	Parking Structure	-Same as above (sub-structure to admin building), revenue source.
7	T-6		Berths 604 and 605	Main container terminal - economic recovery and potential relief effort
8	T-5		Berth 503 - 45,628 sq ft concrete dock on steel piles	Marine revinue source - Economic recovery
9	T-4		Berths 410, 411 - N. end of Slip 3	Kinder Morgan facility - Marine revinue source - Economic recovery
10	T-5		Columbia Grain Facility	$1-$2M Annual revenues (needs refinement for asset value and income)
11	T-6		Hyundai Auto Berth 601 (incl. Hull @ $3M; include walkways, ramps, dolphins, piles, etc.)	Floating RORO facility. Likely most usable facility immediately following event - used for initial relief efforts and auto imports in economic recovery.
12	HIO		Runway 2/20 and 12/30	Secondary airfield for relief effort and access to West side of Willamette River
13	PDX	7111 NE Alderwood Rd	Grd Maint Admin & Shops	Maintenance facility, required to keep PDX operational long term.
14	PDX	7111 NE Alderwood Rd	Grd Maint Facility	Maintenance facility, required to keep PDX operational long term.
15	PDX	7111 NE Alderwood Rd	Grd Maint Facility	Maintenance facility, required to keep PDX operational long term.
16	T-6	7205 N. Marine Dr	Maintenance	Coordination of marine relief and economic recovery activities
17	T-6	7209 N. Marine Dr	Electric Shop Bldg, SW of Admin Bldg	Coordination of marine relief and economic recovery activities

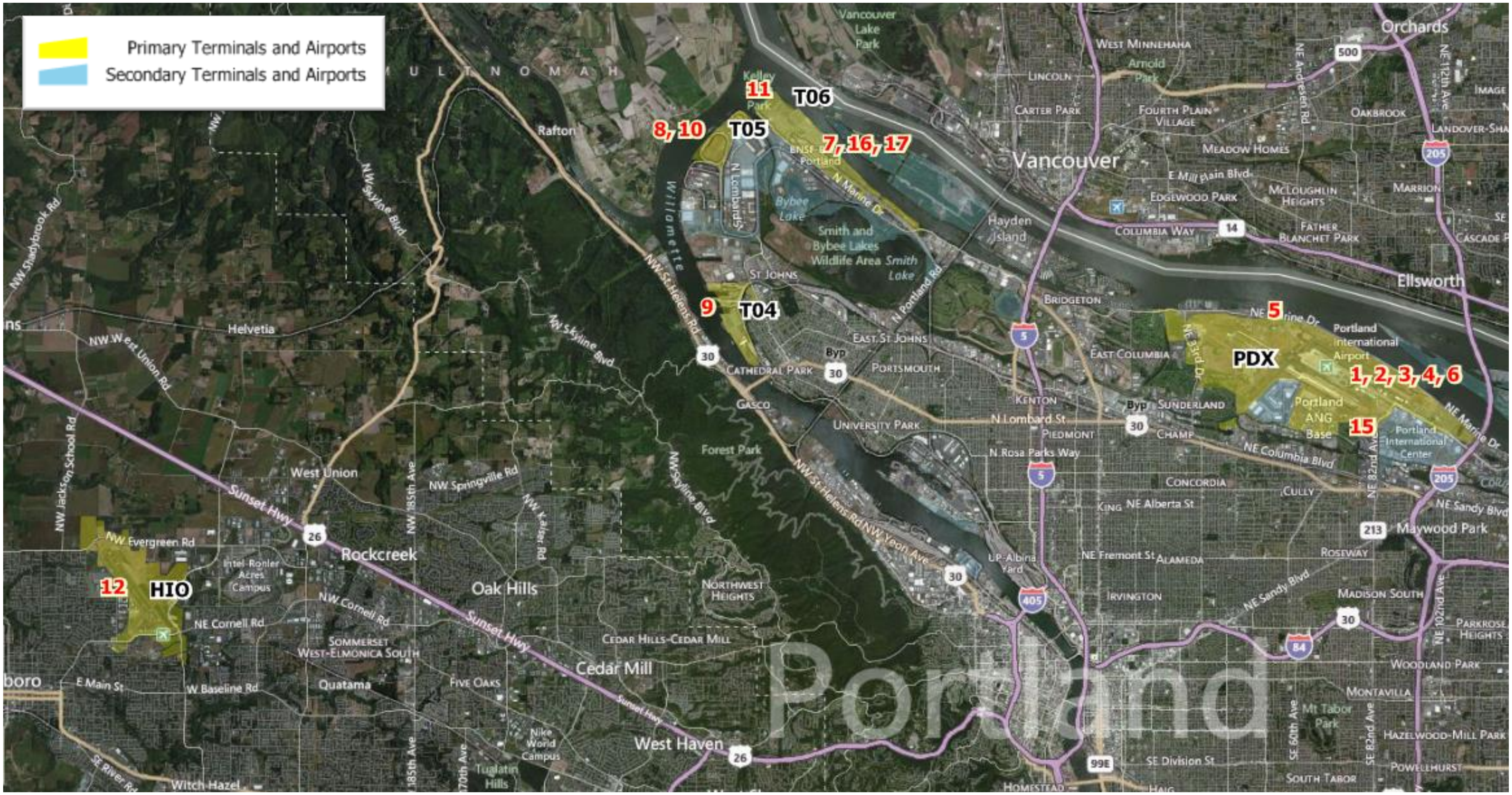


Figure 1: Regional Map of Priority Port of Portland Facilities

Figure 2: Priority Facilities at the Port of Portland International Airport (PDX)

Figure 2 above maps the entire Portland International Airport (PDX), which includes many of the top priority facilities. The top priority facility is the Central Utility Plant building and utility tunnel, providing heating, cooling, hot water, power, and back-up power to the main airport terminal, as noted in Table 1. The selection of the Central Utility Plant as the top priority facility reflects the importance of lifelines and lifeline interconnectivity. The airport is a critical transportation node for the regional economy, and it is pertinent that it have power and back-up power. Secondary importance was given to the primary runway at PDX followed by terminal facilities, the PDX fire station, and the PDX administrative offices/parking structure. The following maps in Figures 3, 4, and 5 detail many of these facilities that are not clear at the scale of Figure 2.

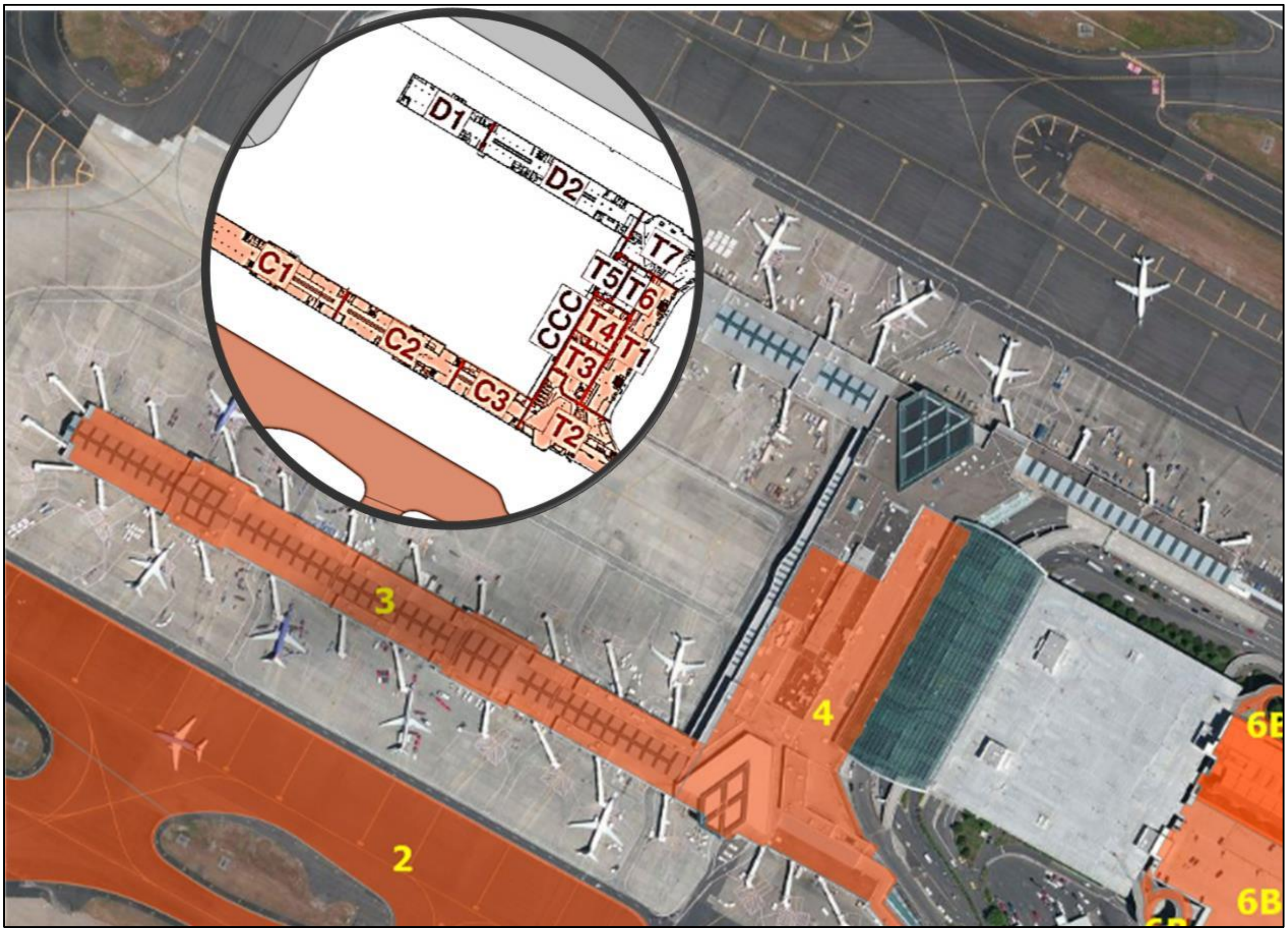


Figure 3: PDX Priority Facilities Detail 1, Terminal Area

Figure 3 above provides a detail of the PDX terminal area with an inset from the Port of Portland CAD maps for clarification. Priority facility number 3 is the C Concourse which includes the CCC communication structure and the EOC, located in section C3 as well as sections C1 and C2 in the diagram. Priority 4 is the main passenger terminal building, and includes T1, T2, T3 and T4 on the CAD inset. Of the priority 4 structure, T2 on the south side is the priority, given the Port of Portland Police Station is within this section of the terminal.

Figure 4: PDX Priority Facilities Detail 2, Terminal Parking and Office Space

Figure 4 provides a detailed view of the PDX facility. This image provides a more detailed view of the top priority Central Utility Plant as well as priority structure 6. Priority structure 6 includes both the parking structure for PDX as well as the administrative offices. Parking (6B), including the onramps, are on floors 1 to 7 and are secondary in importance to office space (6A) on floors 8, 9, and 10. The inset from Bing Maps on Figure 4 provides an oblique view of the structure.

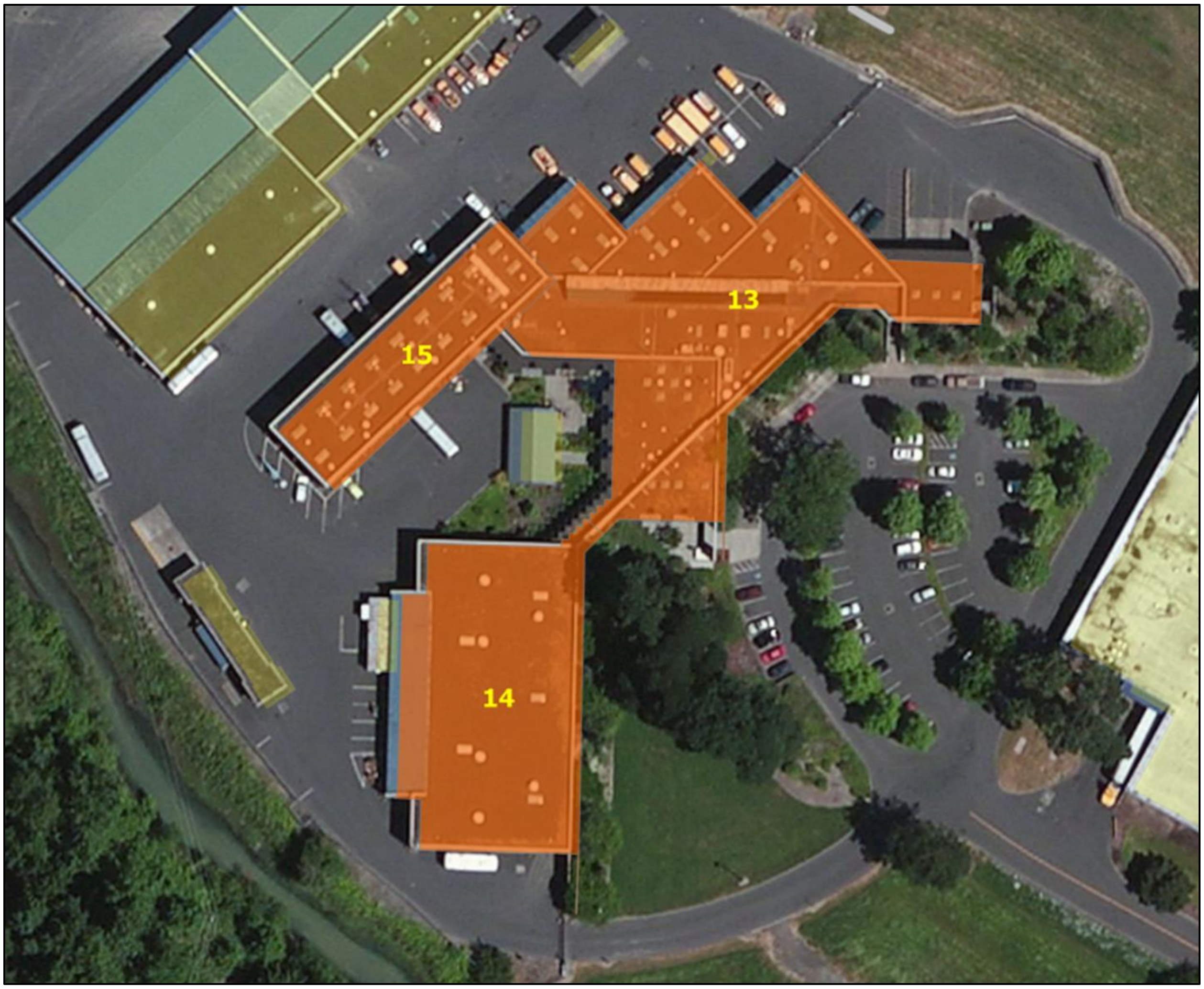


Figure 5: PDX Priority Facilities Detail 3, Maintenance Facilities

Figure 5 above provides a detailed map of the interconnected maintenance and shop facilities comprising priorities 13, 14, and 15. These facilities are required to keep PDX operational in the long term. The demarcation between the interconnected structures can be discerned by the parapets on the roof. For clarification, buildings 14 and 15 are rectangular and building 13 is asymmetrical.

Figure 6: Port of Portland Priority Facilities, Terminal 6

Figure 6 above maps the priority facilities at Terminal 6. Berths 604 and 605 are together priority 7, and the Hyundai Auto Berth is priority 11. These facilities would be crucial for economic recovery following a natural disaster. Given that the Hyundai Auto Berth is a floating roll-on/roll-off facility (RORO), it is anticipated to be the berth most likely to be useable immediately following an event, and is thus key for relief efforts. As evidenced by historic labor disputes at west coast ports, damage to these facilities may have cascading impacts leading to business interruption in the automobile industry, including temporary plant closures.

The maintenance facilities on site are priority 16 and 17 and are used to coordinate marine activities and economic recovery. Terminal 6 is key for the import and export of automobiles and parts.

Figure 7: Port of Portland Priority Facilities, Terminal 4

Berths 410 and 411 are priority 9, and comprise the only priority facility for Terminal 4, the Kinder Morgan facility. Keeping this facility running is a priority for economic recovery as it is a key revenue source.

Figure 8: Port of Portland Priority Facilities, Terminal 5

Figure 8 above maps the key facilities at Terminal 5, including berth 503 (priority 8) and the Columbia Grain facility (priority 10). These facilities generate a considerable amount of income from the import and export of raw materials and are important for economic recovery to the Port.

Figure 9: Port of Portland Priority Facilities, Hillsboro Airport

The primary runway at the Hillsboro Airport is mapped in Figure 9. The runway is ranked priority 12, primarily because it can be used as a redundant airfield if there is considerable damage at PDX. In addition, the Hillsboro Airport has strategic access to the Willamette River.

Hazardous Materials

Figure 10 provides a map of the contaminated sites, brownfields, key facilities, water facilities, and air quality monitoring stations inventoried by the EPA. EPA databases can identify potential disruptions due to offsite damage. The EPA database location precision is often not to the building level, and thus many of the facilities are shown at the entryway to the property. Table 2 is a list of contaminated sites that are included in the EPA database and lie within 100 yards of the Port of Portland facilities (according to the GIS database provided by the Port of Portland). Many of the facilities are associated with the Port of Portland. More details can be obtained by entering the EPA Registry ID after the following URL in a web browser: "*http://iaspub.epa.gov/enviro/fii_query_detail.disp_program_facility?p_registry_id=*" Given the proximity of the Port Facilities to major petrochemical storage facilities, it is important to consider the criticality, vulnerability, and redundancy of these facilities when assessing the lifeline vulnerability of the port.

Table 2: EPA Contaminated or Potentially Contaminated Sites within 100 Yards of Port of Portland Facilities

EPA Registry ID	Primary Name	Address	City	Postal Code
110016646682	End of swan island lagoon	N Basin AVE	Portland	97217
110030826159	Canoe Bay	N Hayden Island DR.	Portland	97217
110037717977	Cenex AG Inc	6135 N Basin AVE	Portland	97217
110037719136	Albers Mill property	1200 NW Naito PKWY	Portland	97209
110037729447	Drums - NE Marine Drive	9000 block, NE Marine DR.	Portland	97211
110037747329	Frontage ditch road spill	NW Frontage RD	Troutdale	97060
110037783477	NW container service	11920 N Burgard RD	Portland	97203-6623
110037783609	Portland airport fire training pits	Off NE Airport Way	Portland	97218
110037786820	Portland ground run-up enclosure facility	5909 NE Mcguire AVE.	Portland	97218
110037788711	Miller Transport	4010 NE Buffalo ST	Portland	97211-2112
110037789060	ODOT - NE Holman and NE Alderwood	NE Holman and NE Alderwood	Portland	97220
110037795972	Port of Portland - Leadbetter	N Leadbetter RD. and N Bybee LAKE CT.	Portland	97217
110037814675	Waterfront Pearl Cond Construction site	1118 NW Naito PKWY	Portland	97209-2818
110037820757	RS Land LLC (Broadmoor GC)	8434 NE 33RD DR	Portland	97211
110037821523	UAL Hangar, Port of Portland	NW Airport Grounds, Marine Drive	Portland	97218
110042136578	Portland Dock Commission	2435 NW Front AVE.	Portland	97209
110042136710	Port of Portland	1260 NW Perimeter WAY	Troutdale	97060-9525

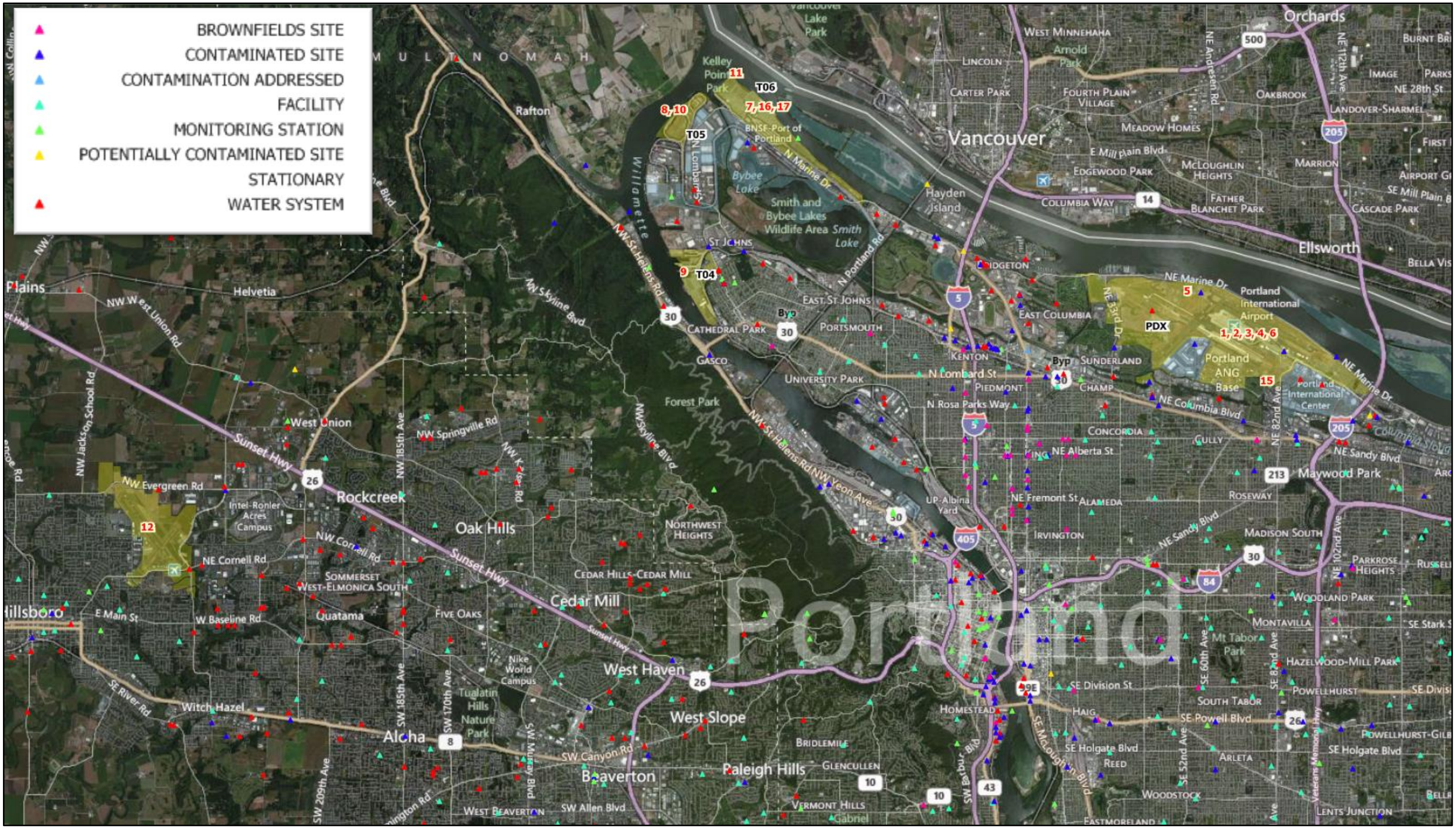


Figure 10: Hazardous Materials in the Portland Region (Source: State of Oregon EPA)

Liquefaction

Liquefaction is the process in which fine-grained, sandy soils lose cohesion due to earthquake vibrations and water saturation. As the ground temporarily exhibits the properties of quicksand, significant damage can occur to structures supported on the ground as well as buried lifelines. Shifting, cracking, and sinking soil causes buildings to settle or tilt, dislodges inflexible pipeline connections, contorts roads and railways- making them impassible, and shifts bridge-decks from their foundations.

Figure 11 presents the regional liquefaction hazard as mapped by Oregon Department of Geology and Mineral Industries (DOGAMI). The DOGAMI liquefaction map estimates the susceptibility using a variety of sources, including recently published lidar-based surficial geology map and published hazards studies. Liquefaction is assigned using susceptibilities as defined by Youd and Perkins (1978) as modified with consideration of Oregon geology and by review of data from Washington (Palmer and others, 2004).

As indicated on the map, most of the facilities of interest are in areas with a high liquefaction potential, with the exception of the Hillsboro Airport which is in an area of moderate liquefaction susceptibility. In addition, since the facilities are collocated along the Willamette and Columbia Rivers, an event resulting in a high water level is likely to impact many of the selected top facilities as well as the lifelines that serve them – including fuel, power, and transportation. Multi-span bridges are particularly vulnerable to liquefaction, as lateral spreading can cause bridge decks to separate from foundations and abutments. Figure 12 maps liquefaction susceptibility along the length of the Columbia River from PDX to the ocean on the Oregon side. Major bridges that cross over the river are vulnerable to either high or moderate levels of liquefaction. It is important to consider that earthquakes that do not directly impact the Port of Portland, such as a major subduction zone event off the coast, could potentially disrupt marine traffic from the Columbia River. In addition, fuel and transportation services that rely on connectivity with the Seattle area could be impacted. Regional impacts of lifeline disruption and lifeline interdependencies should be a key area of investigation for subsequent research into earthquake risk for the Port of Portland.

Figure 13 presents the DOGAMI map for landslide hazard in the Portland region. Many of the areas are designated as Zone VII, but are fairly flat (slope <10 degrees). The zone designation of VII in wet conditions is primarily due to the soils. Table X presents the landslide classification used in HAZUS-MH and implemented by DOGAMI.

Table 3: Landslide Susceptibility of Geologic Groups, HAZUS 2.1 Technical Manual

Geologic Group		Slope Angle, degrees					
		0-10	10-15	15-20	20-30	30-40	>40
(a) DRY (groundwater below level of sliding)							
A	**Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone, c' =300 psf, ϕ' = 35°)**	None	None	I	II	IV	VI
B	**Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone, c' =0, ϕ' = 35°)**	None	III	IV	V	VI	VII
C	**Argillaceous Rocks (shales, clayey soil, existing landslides, poorly compacted fills, c' =0 ϕ' = 20°)**	V	VI	VII	IX	IX	IX
(b) WET (groundwater level at ground surface)							
A	**Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone, c' =300 psf, ϕ' = 35°)**	None	III	VI	VII	VIII	VIII
B	**Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone, c' =0, ϕ' = 35°)**	V	VIII	IX	IX	IX	X
C	**Argillaceous Rocks (shales, clayey soil, existing landslides, poorly compacted fills, c' =0 ϕ' = 20°)**	VII	IX	X	X	X	X

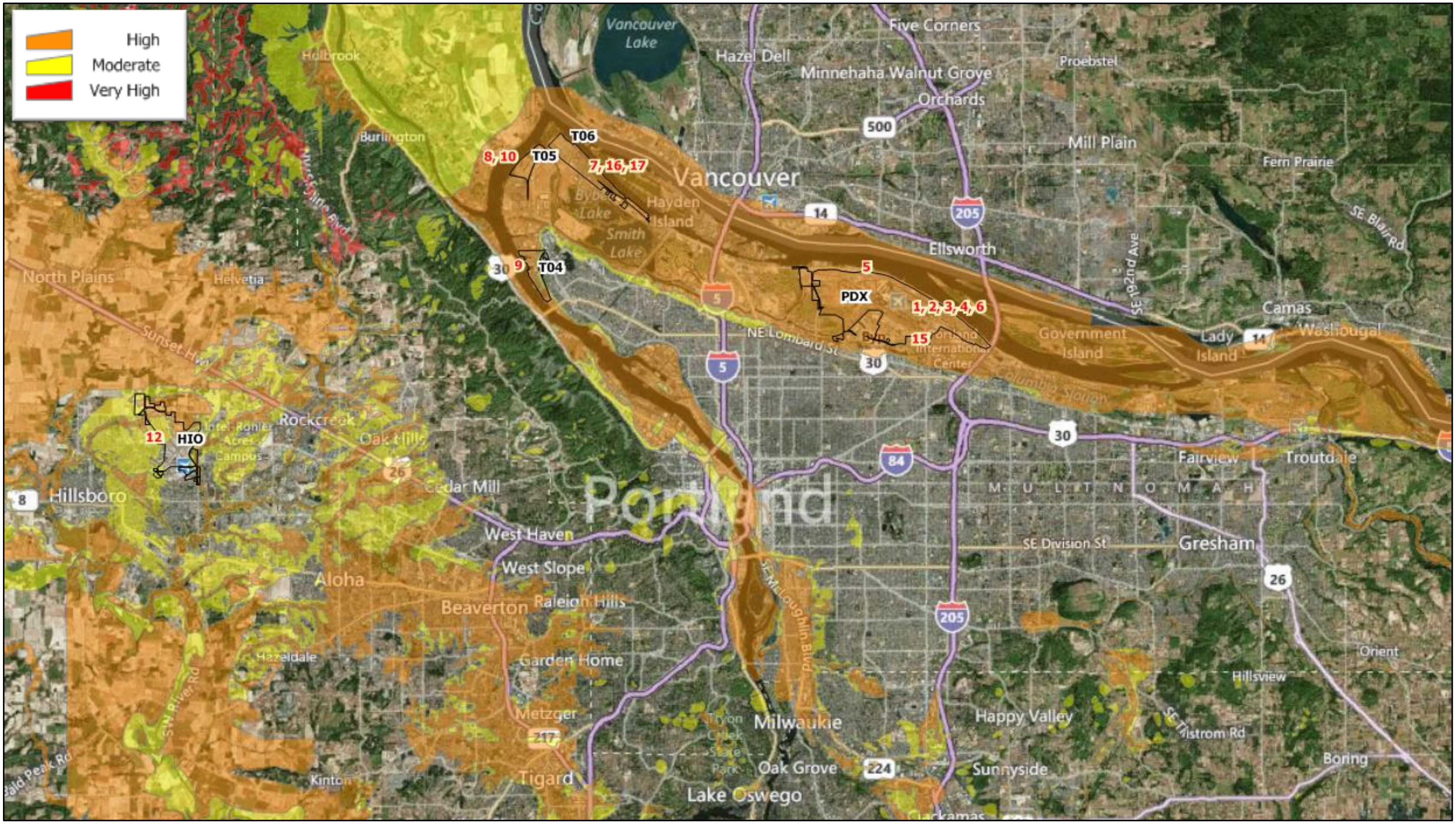


Figure 11: Liquefaction Susceptibility in the Portland Region (Source: Oregon Department of Geology and Mineral Industries, DOGAMI)

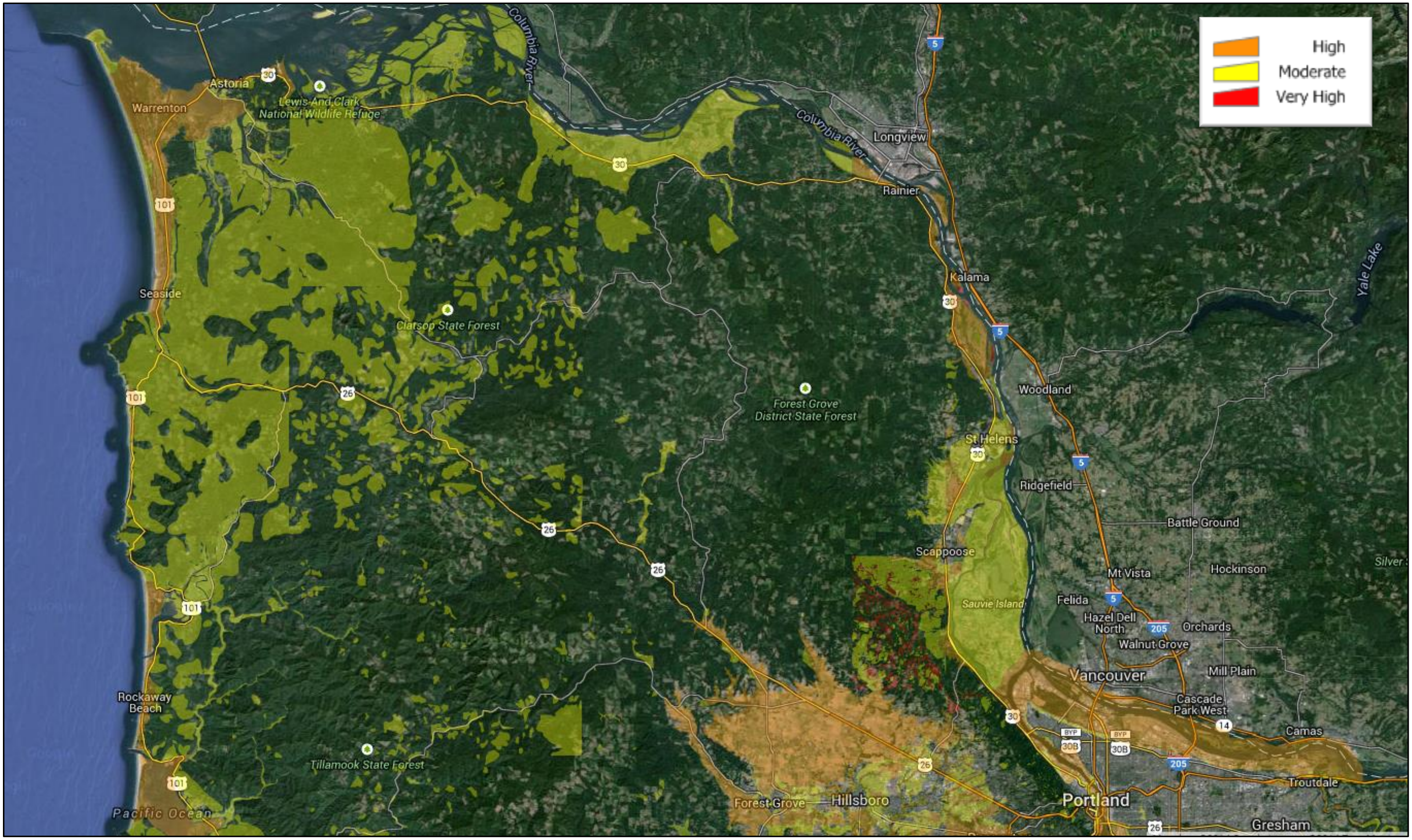


Figure 12: Liquefaction Susceptibility throughout Oregon (Source: Oregon Department of Geology and Mineral Industries, DOGAMI. Note, data does not include the State of Washington)

Figure 13: Landslide Susceptibility Zone in the Portland Region (Source: Oregon Department of Geology and Mineral Industries, DOGAMI)

Lifelines

The bulk of this study has considered potential seismic impacts of the 18 Port of Portland facilities of interest. In addition to the direct risk posed by earthquakes to the Port, there are the potential cascading impacts resulting from damage to the Port affecting services within other parts of the Port or to the wider community. These "secondary impacts" are often difficult to anticipate, but the consequences are often catastrophic and can exceed the direct impacts in terms of economic consequences. Recent examples of secondary impacts include the social disruption resulting from the Fukushima Daiichi nuclear plant meltdown following the 2011 Tōhoku earthquake and tsunami, the inundation in central New Orleans due to the levee breech following Hurricane Katrina, and the business interruption caused by flooding in 7 major industrial parks during the Thailand floods of 2011. Secondary impacts are often triggered by lifeline disruption. Lifelines include electric power, gas and liquid fuel, communications, transportation and water and wastewater systems. Ports themselves are considered a lifeline as part of the transportation system. Massive and potentially lengthy disruptions to the flow of goods and services to and from a region can have devastating effects on a regional economy.

A full assessment of lifeline vulnerability requires a full inventory of the criticality of each of these components to the Port and the regional economic vitality, as well as the vulnerability of each of the individual systems. An individual substation, power plant, pipeline, or crane might be quite vulnerable, but is less of a consideration if there are system redundancies or alternative approaches to resilience (i.e. generators, onsite storage, rail transfer of fuels, etc…). Offsite and regionally, dependencies and alternatives for lifeline services need to be identified, and their vulnerability and dependencies need to be further assessed. Cascading impacts can be planned for, but not without an exhaustive account of the supply chains by which vital lifeline services are supplied, and the anticipated alternatives are supplied.

An assessment of cascading impacts to lifeline vulnerability was outside the scope of this Seismic Risk Assessment Study. This narrative provides a cursory overview of the lifelines, hazardous materials, and regional seismic risk for the Portland area in order to identify potential exposures for further analysis. At the end of this narrative is a suggested approach for assessing lifelines in more detail during a subsequent phase.

Figure 14 presents a regional view of electrical power and fuel pipelines in the region. It is important to note utility data has been provided at a national scale through EIAGIS, and may not be accurate below the level of a few city blocks. Nonetheless, the map illustrates three areas that may be a concern for lifeline disruption: 1) the importance of the Northwest pipeline through the area and through areas of high liquefaction potential at the Port, 2) widespread exposure of refineries, petroleum tank farms and marine petroleum import facilities that could potentially be damaged by an earthquake, and 3) the aviation fuel pipeline that runs through areas of high liquefaction susceptibility. Figures 15 through 17 provide an additional inventory of the water pipelines, gas pipelines, natural gas valves, and gas storage onsite at key locations. All of the mapped pipes, valves, and vaults are in high liquefaction susceptibility areas with the exception of the Hillsboro Airport.

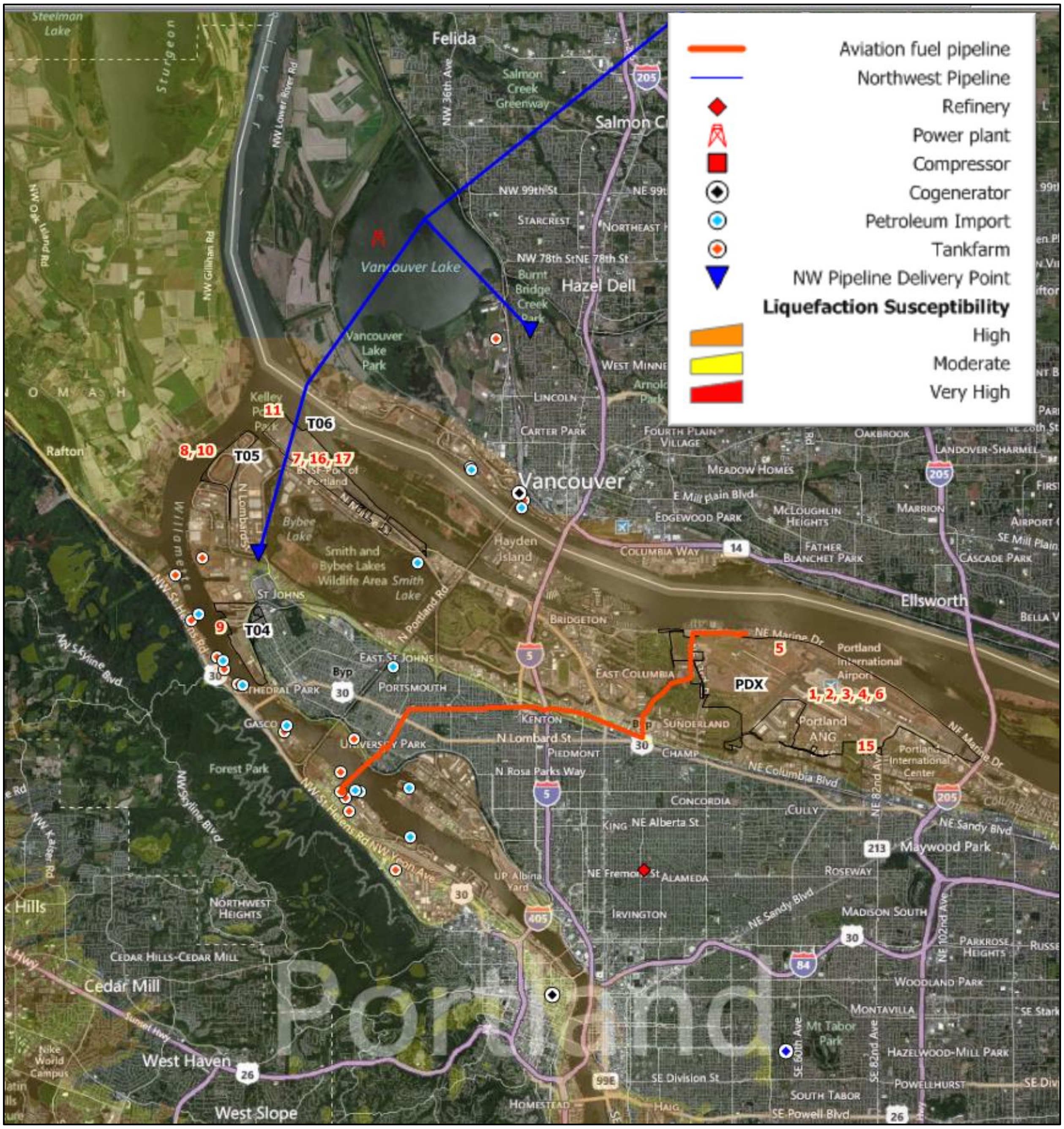


Figure 14: Key Lifelines with Liquefaction Susceptibility in the Portland Region (Sources: Port of Portland, EIAGIS, DOGAMI)

Figure 15: Onsite Natural Gas and Water Pipelines at the Hillsboro Airport (Sources: Port of Portland)

Figure 16: Onsite Natural Gas and Water Pipelines at the Portland International Airport (Sources: Port of Portland)

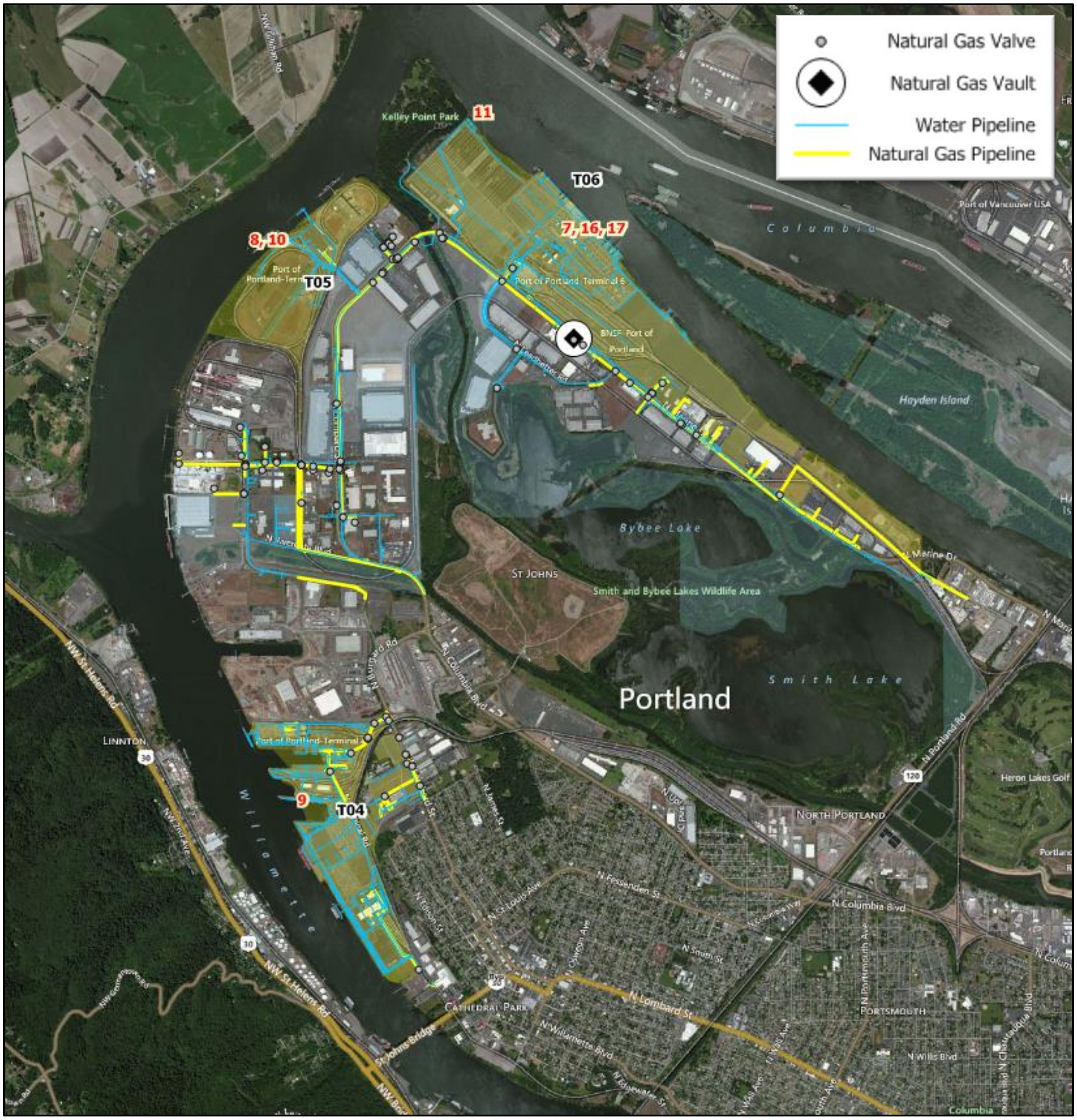
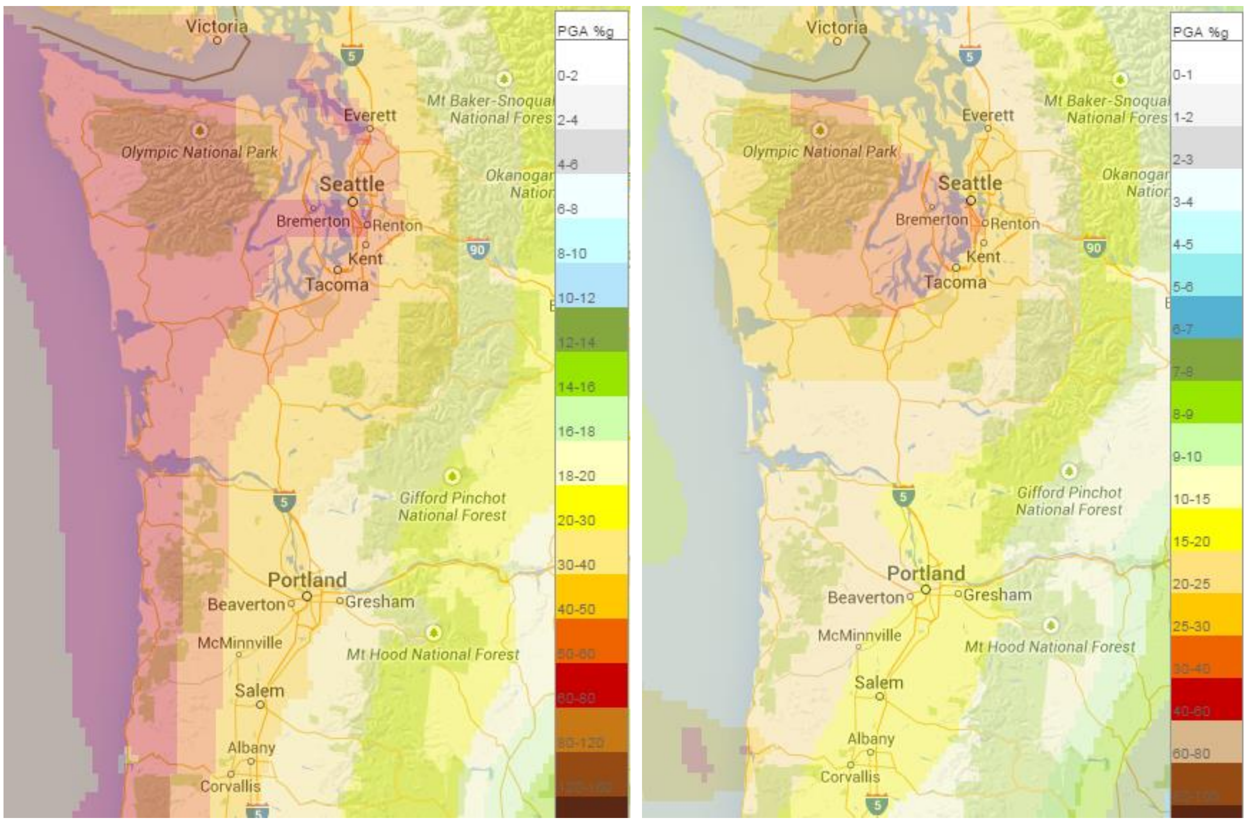


Figure 17: Onsite Natural Gas and Water Pipelines at Terminals 4, 5, and 6 (Sources: Port of Portland)

The Port of Portland aviation and marine facilities provide transportation links that rely upon clear river passage, roadways, railways, communications, back-up power, and other lifelines in order to remain operational. Damage to the transportation lifelines, which include freeways, highways, arterials, ramps, bridges, rail, rail yards and intermodal transportation connectors, can disrupt Port operations by impeding freight flow. Figures 18 and 19 illustrate predicted ground motion and liquefaction potential exposure in relation to transportation infrastructure with the primary routes for freight in the region. As with the major port facilities, much of the area has been identified as having a high potential for liquefaction. Several bridges that cross the Columbia River are exposed to significant levels of liquefaction at the abutments. Dam failure, tsunami inundation, or damage due to ground shaking far from the Port facilities could impact marine traffic for a considerable duration. Thus, it is important to consider earthquakes that do not directly impact Port facilities but may impact freight flow through the port.



(A) **(B)**

Figure 18: 2,475 (A) and 475 (B) Year Probabilistic Peak Ground Acceleration Maps for Western Washington and Oregon (USGS, 2008)

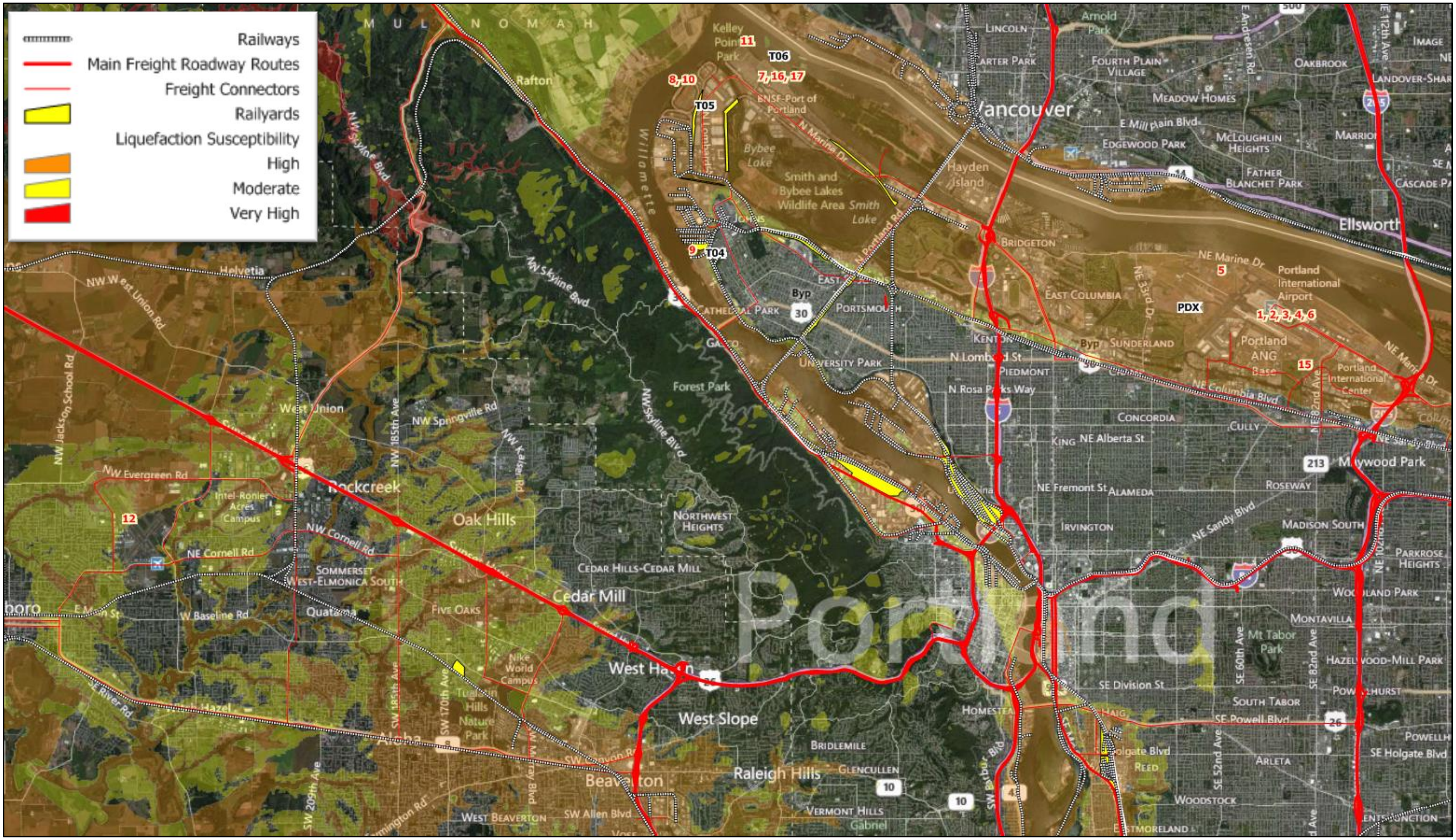


Figure 19: Regional Freight Transportation Lifelines and Liquefaction Susceptibility (Sources: Port of Portland, DOGAMI)

Suggested Approach for Assessing Lifeline Vulnerability

The maps in the foregoing serve as a preliminary illustration of possible concerns with respect to lifeline disruption for the Port of Portland. The Port is dependent upon lifelines to provide basic services, including power, fuel, and clear transportation thoroughfares (roads, bridges, waterways, and rail ways). Communication is also critical to air-traffic, and internet connectivity may be imperative for certain business transactions. All of these lifelines have vulnerabilities. Some lifeline services may depend upon connectivity with distant facilities that may be vulnerable to a large subduction earthquake affecting the coast, or a large event in the Seattle area. Others might depend largely on onsite or local facilities which may have damage that is highly correlated. Redundant routes and back up facilities also have vulnerabilities that would also be considered.

All of these lifelines have interconnections and interdependencies that need to be carefully examined. If backup power is dependent upon the fuel supply, is the fuel supply vulnerable to damage or interruption of supply? Are there possible cascading effects if critical equipment is damaged and requires a long time for repair or replacement? Intermodal transportation links in particular may prove difficult to circumvent. Possible cascading effects should be identified – such as, will there be water for firefighting in the event of extensive pipeline damage? Are there life-safety or environmental vulnerabilities that might need to be considered?

These specific issues may or may not apply to the Port of Portland. A thorough assessment of lifeline vulnerabilities requires an exhaustive cross-lifeline examination of not only hazard exposure, vulnerability and criticality, but dependencies, interconnectivity, and redundancies, which requires the knowledge of Port employees and stakeholders. This section describes a simple process that can be used to stochastically model systemic risk and the potential financial benefits of mitigation. This approach does not quantify environmental or life-safety benefits.

The general approach is to identify and quantify risks and remediation exercises through a series of workshops, data gathering and walk downs, and simulations to find the most severe risks and opportunities for mitigation. Critical to this process is a series of workshops to "brainstorm" possible lifeline risks, usually using a preliminary set of maps such as those included in this section. The aim is to discern potential cascading failures, and remediation options, particularly those within the remit of the Port. In an initial Phase I meeting, as much information concerning the possible risk is gathered. This data establishes the basis for data collection, which is followed up by a Phase II meeting to clarify any missing information. Once the data is available, a system network is developed and used in a series of analysis to model the final probability of risk given dependencies. These results are combined with the remediation options to yield both the cost effectiveness of remediation and a ranking of risks.

The following is a description of each phase of lifeline risk identification process:

Step 1: Phase 1 Hazard Screening

The first step is to develop a series of basic exposure and hazard maps such as those presented in this section. This provides a basis for discussion in the stakeholder meetings, and is often the basis of data collected in task 3. It is also advised to have financial impact estimates for outages of a given duration by facility.

Step 2: Phase I Stakeholder Workshops- Risk and Remediation Identification

The Phase I stakeholder workshop is a critical brainstorming session. Those familiar with risk, onsite facilities, and lifelines develop a long list of possible system impacts that could disrupt the Port. For each risk, consequences and interdependencies must be identified and quantified in order to run the risk assessment. For example, an engineer may posit – if equipment X were to fail, facility Y would not be operational. Equipment X depends on utility Z. We have 2 weeks onsite service available if service were to be interrupted, but it might take 4 weeks to restore service. These numbers are generally debated, and a mean value is taken to establish: 1) the financial consequences of the risk, 2) remediation opportunities, and 3) the reduced consequences given remediation. In addition, the likelihood of each failure – and any dependencies – must be quantified. Although this might not be possible during the Phase I workshop, likelihood estimates should be collected where feasible.

Requirements:

- Risk likelihoods (%) and consequences (in dollar amounts) of teach specified risk
- Interdependent risks and likelihoods (%) (i.e., risks of each link in a chain)
- Existing controls currently in place to minimize the risk
- The adverse consequences of the risk before remediation
- Possible remediation plans with associated costs
- Each risk's adverse consequences and likelihoods should the new treatment plan be implemented

Step 3: Exposure Development

Exposure development is the process of using the information gathered in Step 2 and transforming it into a data format suitable for risk assessment. The process will generally start with exposure maps such as those depicted in Figures 14 and 19, but generally must be abstracted to form an intermodal system model for risk assessment. In many cases, the exposure data must be developed at several scales – and might draw upon system network assessments from multiple studies and reports. For example, it may not be reasonable to assess widespread bridge collapse on the Columbia River as part of a lifeline study for the Port, but given the risk it may be feasible to review ODOT seismic studies or contact bridge engineers to quantify the probability of bridge collapse and the expected duration that marine cargo might be affected.

Exposure development must be conducted in concert with onsite reviews of potential risks, and extensive interviews with facility managers to classify the vulnerability of key components. This might require reviewing anchoring and bracing, pipeline connections, or other minute details that are identified as key factors in Step 2.

Step 4: Phase II Stakeholder Workshop- Data Clarification and Expert Assessment

In the Phase II stakeholder workshop, findings from Step 3 are discussed and the project team identifies data gaps that must be filled through expert opinion. In many cases, this is the only feasible way to characterize certain key risks, and likelihoods and consequences must be carefully considered.

Step 5: Risk Analysis

A Monte-Carlo risk assessment integrating dependencies and earthquake hazard is used to quantify the economic consequences of failure and the effectiveness of the proposed remediation efforts.

Step 6: Final Report and Workshop

The results of Steps 1-5 are documented in a report and presented to stakeholders in a final workshop.

Given that lifeline risk is a complex regional problem with multiple stakeholders, it is preferable to include as many lifeline stakeholders as possible so that risks can be accurately identified and quantified. Although remediation that the Port can directly fund may be limited to Port facilities, identification of potential cascading impacts may justify hardening of external facilities, which might be eligible for state or federal funding.