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Seismic Hazard Assessment of Oregon Highway Truck Routes

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OTREC

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TRANSPORTATION
RESEARCH AND
EDUCATION CONSORTIUM

Seismic Hazard Assessment of Oregon Highway Truck Routes

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SEISMIC HAZARD ASSESSMENT OF OREGON HIGHWAY TRUCK ROUTES

Report

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by

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16. Abstract

This research project developed a seismic risk assessment model along the major truck routes in Oregon. The study had adopted federally developed software tools called Risk for Earthquake Damage to Roadway Systems (REDARS2) and HAZUS-MH. The model was the first time REDARS2 has been adopted and used in research outside of the original development team, presenting a number of unique challenges. The development of the model was a complex, intensive process that required a significant research effort, manipulation and adjustment of data. Furthermore, limitations of the software tools themselves had been identified that prevented the inclusion of important aspects such as liquefaction induced damage and refinement of the transportation network.

The main objective of this research were to refine the data from a first generation of the model to more realistically represent the bridge inventory, to address the seismicity of the Pacific Northwest, conduct sensitivity analyses of soil data on the analyses results and develop a seismic network model of Oregon bridges for purposes of assessing the seismic vulnerability of roadway segments.

The first generation model relied on default settings within the program to determine the economic loss due to repair and replacement of damaged bridges. The assumptions used in the analyses have been reviewed and Oregon specific data was incorporated for the model. The largest earthquake now considered to be at a highest level of probability in the Pacific Northwest is a subduction zone earthquake. The major shortcoming of REDARS2 is its inability to incorporate the subduction zone attenuation relationship into the analysis. To incorporate that capability into the model, shakemaps were developed by USGS for Cascadia subduction zone scenario events and incorporated as the demand on the refined model.

Analyses of the transportation network incorporating bridge routes and post processing of the data with input from Oregon DOT bridge engineers resulted in recommendations toward bridge route priority strategies. The majority of the bridges that indicated the possibility of damage were types associated with multi-column bents, simply-supported concrete superstructures and simply-supported steel superstructures. Of the major highway routes that were considered, I-405, section of I-5 (from Multnomah to Clackamas Counties), I-84, I-205 and US-101 were the top five on the preliminary priority for seismic retrofit. These routes need to be analyzed more and advanced cost-benefit investigations should be done before retrofit decisions are made.

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DISCLAIMER

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EXECUTIVE SUMMARY

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1.0 INTRODUCTION AND PROJECT DESCRIPTION

1.1 INTRODUCTION

Geologists have indicated that the question is not if a catastrophic earthquake will occur in Oregon, but when one will occur. Seismologists have long known about the potential earthquake threat in the Pacific Northwest stretching from northern Vancouver Island in Canada to northern California which is the Cascadia subduction zone, where one giant plate of the Earth's surface is diving deep beneath another one.

The effects of an earthquake of this magnitude can result in potential sudden detrimental impact on the transportation infrastructure where bridges represent vulnerability points within the network. When bridge damage occurs during a seismic event, short-term or long-term interruptions to traffic flow result. This will delay emergency response in the hours after the event, and restrict the movement of people and goods for months. Hence, the economic impact of bridge damage includes not only the cost of structural repair, but also longer term consequences relating to valued loss of time when commuter and freight travel slows down to navigate the disrupted network.

Hundreds of bridges in the State of Oregon are still vulnerable to earthquake damage. Over the last decade it has been shown during the course of bridge inspections that many of the bridges are showing signs of deterioration. As reported by Patrick Brennan for the Oregon Legislative Committee Services council in a brief, the causes of this deterioration problem include older construction methods, structures beyond their intended construction life, and a scale of increased use that was not accounted for in the original design of the bridges. Currently, the Oregon Department of Transportation (ODOT) owns and maintains just over 2600 bridges distributed over the state controlled routes shown in Figure 1. Of the approximately 2600 bridges, a fifth of them are beyond the 50-year construction life (Brennan 2004).

The risk associated with earthquake hazards on highway systems is largely dependent on the complexity and redundancy of a network in providing smooth traffic flow. Seismic Risk Assessment (RSA) studies can provide decision makers with an appreciation of the importance of having a highway network resistant to earthquakes and information to make the network invulnerable to these events. The main objective of this research project is to address the major limitations for the current state of the model to more appropriately represent the traffic conditions and the seismicity of the Pacific Northwest. This involved refining the transportation network, bridge database and more appropriately studying the region and investigating the sensitivity of key input parameters such as bridge fragility, damage threshold and liquefaction threshold on the global results. The model was then used to analyze the network resulting in recommendations toward bridge retrofit strategies.

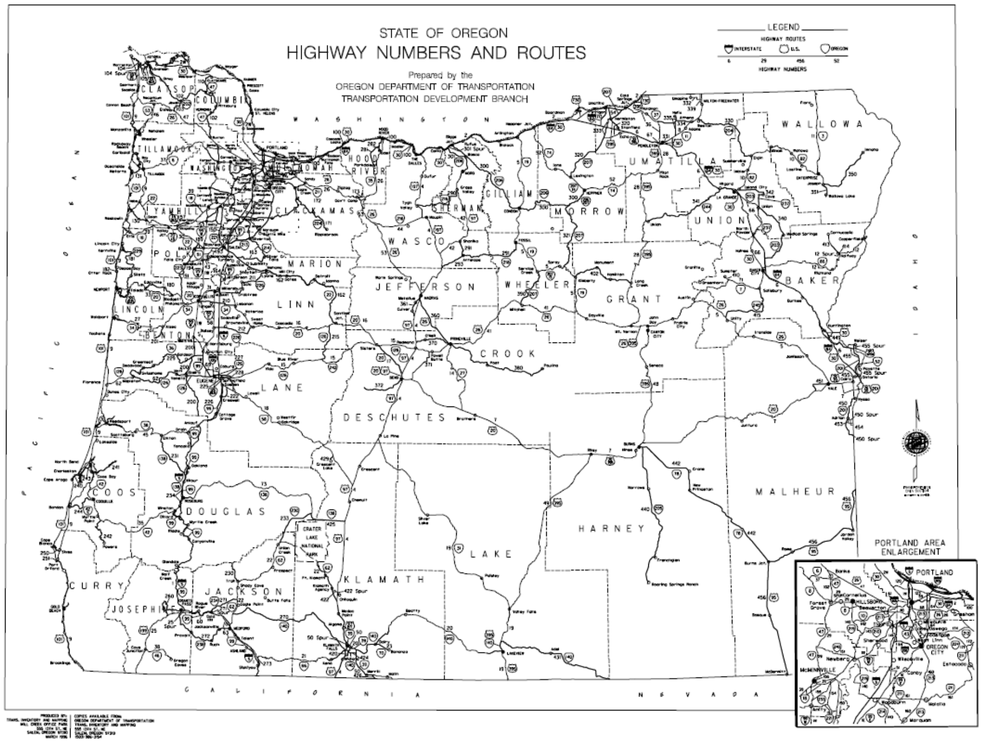

Figure 1: Map of Oregon Department of Transportation State Routes (ODOT 2006)

1.2 OBJECTIVE

The primary objective of this research is to develop seismic network model of Oregon bridges for seismic retrofit prioritization, and to assess the seismic vulnerability of roadway components. This project builds up on the already created GIS model of the roadway and bridge network using new technology developed for the Federal Highway Administration called REDARS2 (**R**isks from **E**arthquake **DA**mage to **R**oadway **S**ystems) by Portland State University (Dusicka 2008). In the previous study, a number of limitations in the existing data as well as the software capabilities itself had been identified to the point that without further research and implementation, the results of the existing model are not realistic.

The objectives of this research are therefore to:

- Implement subduction zone attenuation relationship as part of the analysis capabilities of REDARS2
- Verify analysis results and conduct sensitivity analyses for key input parameters
- Refine the statewide traffic data to more closely reflect current condition
- Assess vulnerability of the existing network and develop recommendations for bridge retrofit.
- Compare REDARS2 vs. HAZUS-MH

1.3 STUDY AREA

The previous study done by PSU of the seismic vulnerability assessment has been on bridges lying on or crossing over Oregon highway routes in the area that included all highway routes lying inside or west of the I-5 corridor, highway routes in the Portland area, the entire length of US-101 and a partial I-84 Columbia River Highway. The bridge data were collected to include bridges up to the year 2008.

The previous model was further refined to more closely reflect current condition and include subduction zone considerations and also verify analysis results and conduct sensitivity analyses for key input parameters. The next step in model development is to include a more enhanced transportation network beyond highways and include county owned bridges and to encompass the entire state of Oregon (Figure 2).

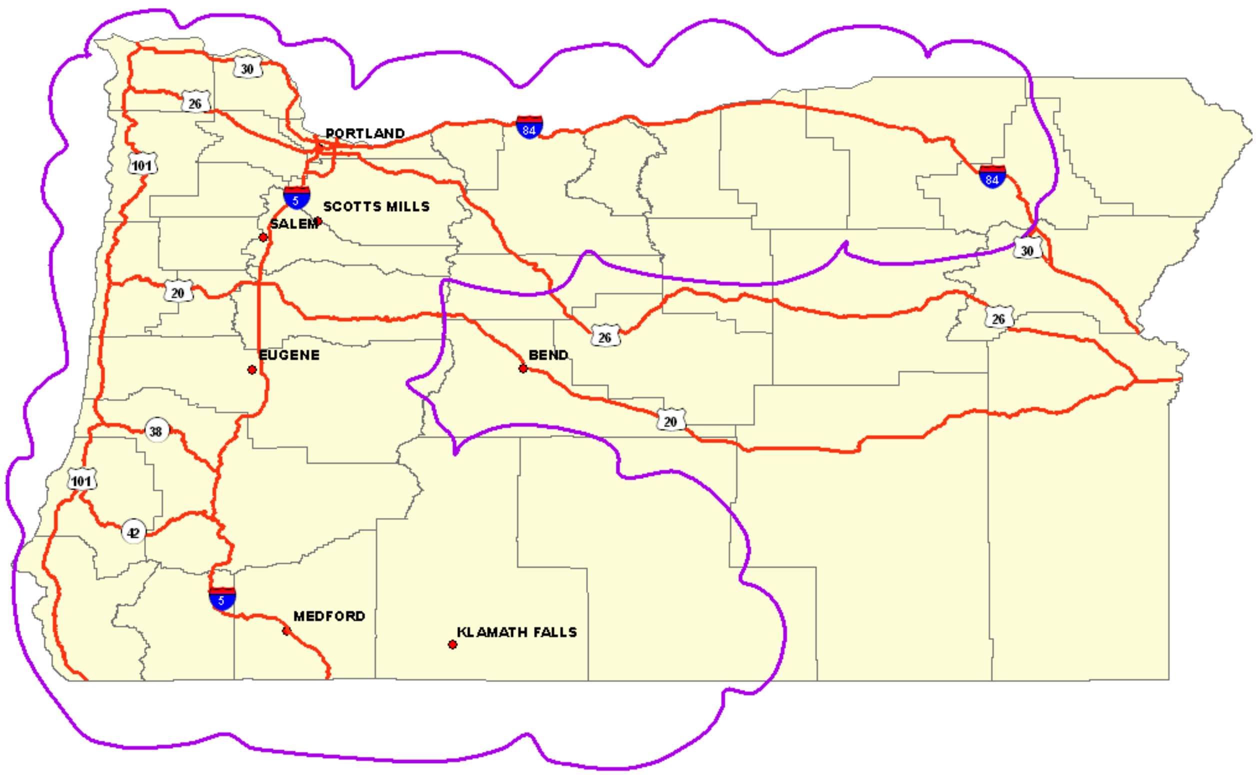

Figure 2: Study Area

2.0 BACKGROUND

2.1 EARTHQUAKE HAZARD ASSESSMENT

The earthquake hazard assessment provides local, state and regional officials with a decision support tool for estimating potential losses from scenario earthquakes. Being able to estimate this gives users that capability to anticipate the consequences of future earthquakes and to develop plans and strategies for reducing risk. The Seismic Risk Analysis (SRA) methodology is a synthesis of models developed by earth scientists, geotechnical and structural earthquake engineers, transportation engineers and planners, and economists. The methodology can develop multiple types of results from deterministic or probabilistic approaches and from local to large geographic areas. Such results can be developed for use in pre-earthquake assessment of various options for seismic risk reduction after an actual earthquake. The software products utilized in this study are HAZUS-MH and REDARS2. The results from REDARS2 have been compared to HAZUS-MH to verify REDARS2 analysis results.

2.2 REDARS2™

REDARS2™ 2 (**R**isks from **E**arthquake **DA**mage to **R**oadway **S**ystems) is GIS software that is used to develop deterministic and probabilistic estimates of the seismic performance of highway-roadway systems. The methodology and software was the result of 12 years of development with the financial sponsorship of FHWA. REDARS2™ 2 uses a default model for estimating bridge damage due to ground motions that corresponds to the HAZUS99-SR2 model (*FEMA 2002*), which is the earliest version of HAZUS-MH. However, REDAR2 also has the capability to conduct roadway transportation network analysis. Seismic performance of these systems is measured in terms of potential for earthquake-induced disruptions of system-wide travel times and traffic flows, and the economic impacts and other losses due to these disruptions.

2.3 HAZUS-MH

HAZUS-MH (Hazards U.S.) is also GIS software used for loss estimation application. GIS technology facilitates the manipulation of data. HAZUS-MH is a methodology that has been developed for the Federal Emergency Management Agency (FEMA) by the National Institute of Building Sciences (NIBS) to provide a tool for developing earthquake loss estimates for use in anticipating the possible nature and scope of the emergency response needed to cope with an earthquake-related disaster, developing plans for recovery and reconstruction following a disaster, and mitigating the possible consequences of earthquakes (FEMA 2003).

Interdependence of components on overall system functionality is not addressed by HAZUS. Such considerations require a network system analysis that would be performed separately by a highway system expert. HAZUS-MH methodology, however, provides a series of combination attenuation relationships including a Subduction Event.

2.4 PREVIOUS RESEARCH ON SEISMIC VULNERABILITY ASSESSMENTS

Seismic Design Decision Analysis (SDDA) is a methodology that was introduced by Whitman et al. in 1975. Most seismic risk assessments (SRA) that are being used have been based on this methodology. SDDA considers the effects of earthquake hazard, damage, and also economic losses. The effects of the damages were studied as probabilities of different damage levels.

The vulnerability of components to earthquake damage depends on the seismic capacity of the component together with the earthquake hazard. King et al. (1997) discuss the advantages of creating generic classes where components can be grouped. This makes it possible to predict, deterministically, the relationships for each class that quantify the damage as a function of the ground shaking.

Earthquake damage to highway components can go well beyond life safety risks and the costs to repair the component itself. When bridge damage occurs during a seismic event, short-term or long-term interruptions to traffic flow result. This can impact post-earthquake emergency response, repair and reconstruction. The level of impact depends on the seismic performance of individual components and the characteristics of the highways system such as network configuration, location, redundancy, traffic capacity and traffic volume (Werner 2006).

Werner and Taylor (2002) emphasized the significance of observing component functionality and location within a lifeline system to assess system performance. Component functionality depends on seismic response characteristics of a component and the state of damage, and also how the damage can be repaired, cost of repair and its significance in the overall system. Knowing whether a bridge will be fully closed, partially open, or fully open provides a means of analyzing networks as a whole.

One of main end results from SRA of roadway systems is the estimation of economic impacts of earthquake damage to the system. Recent studies done on transportation networks place a strong emphasis on indirect costs due to traffic flow and travel times. Indirect economic loss estimate due to damaged bridges within the highway system from an earthquake event for Saint Louis was performed by Enke et al. (2008). Their results showed that the indirect loss is significant when compared to the direct loss resulting from bridge damage.

The scope of seismic risk assessment gets larger as new methodologies for seismic risk analysis that provides a basis for developing mitigation plans and policies, emergency preparedness, and response and recovery planning are accessible. Risk assessment software tools like HAZUS and REDARS2 result in estimates of hazard-related damage and loss estimates before, or after, a disaster occurs. Recent studies conducted on transportation networks place a strong emphasis on indirect costs due to traffic flow and travel times. A study by Stevanovic and Nadimpalli (2010), presents the impact of degree of damage on the traffic in terms of user delay costs and determine how the earthquake damage influence traffic in terms of AM peak, mid day, PM peak, and off peak traffic. The study found some links that are susceptible to damage on one scenario are critical in carrying detour traffic to other scenario. The cost estimate indicated that the maximum impacts would be imposed on PM traffic.

Dusicka et al. (2008) developed a GIS model of the roadway and bridge network using new technology developed for the Federal Highway Administration called REDARS2. The research project concentrated on the development of a strategy to prioritize bridges on Oregon's freight routes for seismic retrofit. The data compiled has provided a strong foundation to this research. The following chapters will present a comparable SRA of the bridges in the Oregon highway network given various ground motion hazards.

2.5 APPLICATION AND FUTURE USE OF OREGON REDARS2 MODEL

The Oregon REDARS2 model can be useful for both pre-earthquake planning and post-earthquake response. Before an earthquake occurs, it can be used to assess the transportation network and formulate a plan for reduction of seismic risk. Different retrofit strategies and prioritizations can be assessed and weighed against each other, and routes of strategic importance or of high seismicity can be identified.

After an earthquake, the model can be used to assist emergency response in numerous ways. For example, it can estimate potential locations within the network where damage is likely to occur and assist with planning rerouting strategies. Potential traffic flow bottlenecks can be identified, and strategies can be formulated for prioritizing bridge or pavement repair following the event.

Further potential applications of the highway network model are detailed in Chapters 1 and 2 of the REDARS2 Technical Manual (Werner 2006). The software is capable of a number of different types of probabilistic and deterministic output that can be interpreted in a variety of ways to assist in seismic risk reduction decision making. Future research and use of the developed model and program will enhance the understanding of bridge seismic vulnerability and emergency response planning in Oregon. This will provide decision makers with some of the tools necessary to assess and appropriately address the weakness of the nation's transportation infrastructure.

2.6 BRIDGES CONSIDERED IN THE ASSESSMENT

In total, the study area includes over 1900 bridges. Over 1200 of these bridges lie on major Oregon routes. Table 3 gives a breakdown of the distribution of the bridges on major Oregon routes. Notably, 502 bridges, or 36% of the bridges considered, lie on Interstate 5, generally considered one of Oregon's major routes as the highway connects to California and Washington states. Table 4 breaks down the predominant types of material of the bridges considered in the assessment.

The NBI database is not a complete description of each bridge. However, it provides sufficient information to allow for general classification. The bridges in this study are classified based on their construction material (Table 1), construction type (Table 2) and the number of spans. This information is contained in three of the 116 fields in the NBI.

Table 1: Construction Materials Listed in NBI (*FHWA, 1995a*).

Description	
Concrete	Prestressed Concrete Continuous
Concrete Continuous	Wood or Timber
Steel	Masonry
Steel Continuous	Aluminum, Wrought Iron, or Cast Iron
Prestressed Concrete	Other

Table 2: Construction Types Listed in NBI (*FHWA, 1995a*).

Description	
Slab	Suspension
Stringer/Multi-beam or Girder	Stayed Girder
Girder and Floor beam System	Movable - Lift
Tee Beam	Movable - Bascule
Box Beam or Girders - Multiple	Movable - Swing
Box Beam or Girders - Single or Spread	Tunnel
Frame	Culvert
Orthotropic	Mixed Types
Truss - Deck	Segmental Box Girder
Truss - Thru	Channel Beam
Arch - Deck	Other
Arch – Thru	

Table 3: Distribution of bridges on major routes

Routes	No of Bridges
I-5	502
I-84	200
US-101	143
US 26	76
I-205	82
I-405	56
US-30	38
US-20	32
OR-38	18
OR-42	54

Table 4 evaluates the predominant types of design of the majority of bridges considered in the assessment. 1053 (54%) of the bridges considered are of stringer/multi-beam design; 340 (17%) are slab designed bridges and 310 (16%) are multiple box beams or girders.

Table 4: Predominant Bridge classes and their proportion

NAME	NUMBER	%
Concrete continuous Stringer/Multi-beam or Girder	488	25 %
Prestressed Concrete Stringer/Multi-beam or Girder	253	13 %
Prestressed Concrete Slab	183	9 %
Concrete Continuous Box Beam or Girders – Multiple	148	8 %
Concrete Continuous Slab	117	6 %
Steel Stringer/Multi-beam or Girder	106	5 %
Prestressed Concrete Continuous Box Beam or Girders – Multiple	92	4 %
Concrete Stringer/Multi-beam or Girder	85	4 %
Prestressed Concrete Box Beam or Girders – Multiple	70	4 %
Prestressed Concrete Continuous Stringer/Multi-beam or Girder	64	3 %
Steel Continuous Stringer/Multi-beam or Girder	55	3 %
Concrete Slab	38	2 %
Steel Truss – Deck	38	2 %
Other	201	10 %

Multi-column bents and simply-supported concrete superstructure and simply-supported steel superstructure are the most susceptible for damage. As can be seen on Table B. 1 (APPENDIX B) the median ground motions leading to onset of damage states are the lowest compared to the other types of structures.

Table 5: Bridge Types and number of spans

Bridge Type	Single Span	Multi Span
Stringer/Multi-beam or Girder	81	970
Box Beam or Girders – Multiple	51	289
Slab	124	218
Truss – Deck	17	75
Girder and Floor beam System	0	31
Truss – Thru	2	22
Frame	25	13
Box Beam or Girders - Single or Spread	4	12
Tee Beam	1	2
Other	0	1

Figure 3 itemizes the year construction was completed of each of the considered bridges in the model. The figure shows that only 609 (31%) of the bridges were constructed after 1970. The age of construction of the bridges is especially important when assessing seismic vulnerability because little consideration was given to seismic resistance prior to the San Fernando earthquake of 1971 (Roberts 1991). Further, bridges completed before 1960 are beyond or near the end of the 50-year design life.

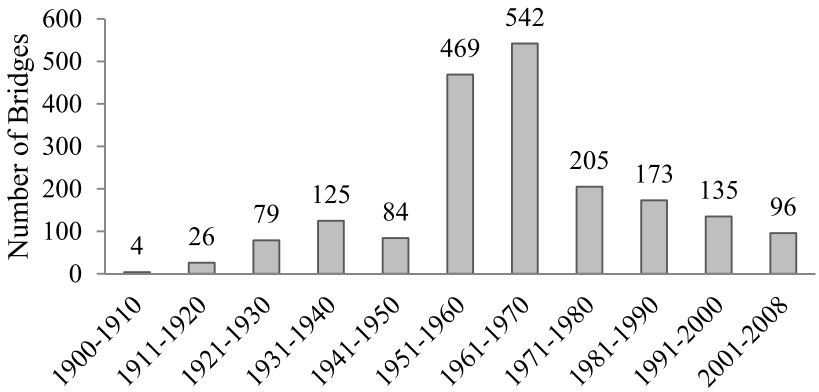

Figure 3: Distribution of year of construction completion

3.0 REFINEMENT OF OREGON REDARS2 HIGHWAY NETWORK MODEL

3.1 REDARS2 BRIDGE MODEL

Research has been conducted by Portland State University of the Oregon bridges for seismic vulnerability assessment in 2008. The study had succeeded in investigating, compiling and manipulating data across multiple civil engineering disciplines to create a dynamic, functional and modifiable GIS model of the network of Oregon freight routes vulnerable to regional seismicity (Dusicka 2008). The results from that study showed a number of limitations in the existing data as well as the software capabilities of REDARS2 had been identified to the point that without additional research and implementation, the results of some aspects of the model were not realistic. Inaccuracies in the soil data and skew angles were identified and corrected.

3.1.1 Soil Data

Available structural and geotechnical drawings containing soils information for bridges in the study area were analyzed. The drawings were obtained from ODOT electronic and paper archives. Some of the errors in the soil data were blow count numbers. And for bridges with available borehole data, the soil properties recorded were further analyzed and rectifications made whenever errors were encountered.

Bridges were assessed as potentially liquefiable or non-liquefiable, and the geotechnical information was updated for bridges in the study area. The initial approach for assessing liquefaction for the respective bridges located within the study region was to screen for liquefaction potential based on the geological sedimentary deposits, water table depth less than 15m (49ft), evaluation of sensitive clays (Only UCS soil types CL or ML and AASHTO soil types A-4, A-2-4, A-6, and A-2-6 meet these criteria.), and final soil classification. However, since ODOT's Bridge Data System (BDS) did not contain any of the relevant geological data to eliminate any of the structures based on geological composite, the soils profiles were initially only eliminated relative to the water table depth.

Soil survey data which are a product of the National Cooperative Soil Survey, from the USDA website were downloaded and used for screening for liquefaction potential. These soil data maps include water table depth and engineering soil properties such as soil classification according to unified soil classification (UCS) and AASHTO and soil classification based on percentage of clay and liquid limit (LL). Oregon has 39 soil survey areas and each survey area has been mapped at different scales and different levels of detail. Therefore, the map unit symbols, soil properties and interpretations are not compatible completely across the soil survey boundaries which made the task laborious.

3.1.2 Skew Angles

The NBI defines skew angle of the bridge, in degrees, between the centerline of a pier and a line normal to the roadway centerline as shown on Figure 4. For a right bridge with no skew, ANGLE = 0. If the bridge is curved and has a variable skew, the average skew is recorded. Sometimes the NBI database had shown that ANGLE = 99 degrees, which signifies a major variation in skews of the substructure units across the length of the bridge. However, in the NBI data there were inconsistencies found in skew angle definitions. To verify the accuracy of the skew angles given in the NBI, first the bridges having a skew angle of greater than 45 degrees were filtered out. For those bridges with a drawing available, the values were compared for consistency. According to the NBI, of the 1938 bridges, 77 bridges have skew angles greater than 45. Of these 27 were inconsistent with the REDARS2 definition of skew angle, 14 were consistent and 35 were missing drawings. The list of these bridges is given in APPENDIX E. The skew angles for the 27 bridges have been updated and included in the current model.

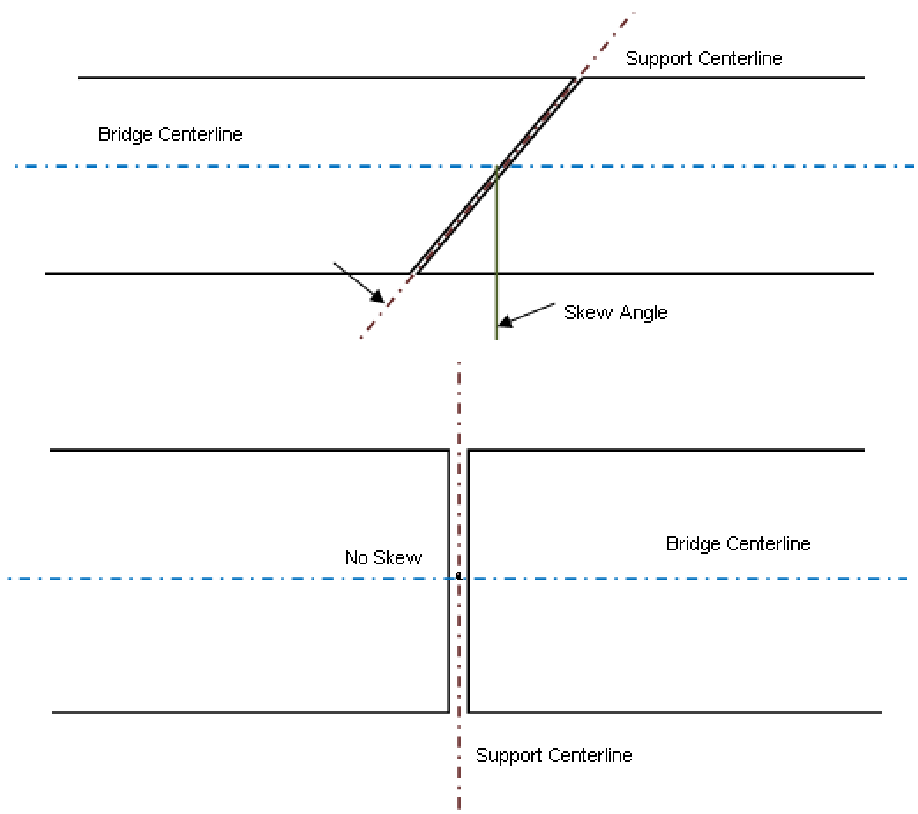

Figure 4: Schematic Depicting Bridge Skew Angle.

3.1.3 Replacement and Repair Cost

REDARS2 uses default models for estimating bridge damage due to permanent ground displacement, as well as post-earthquake repair costs, downtimes, and traffic states as a function of the bridge's damage state. But it also gives the user an option to implement user-defined values to override the default REDARS2 data. In the default repair model, the repair cost is computed as the product of a repair cost ratio (*RCR*) which depends on the bridge's damage state, and the replacement cost (REP = \$150/ft^2 x Deck Area), which depends on the bridge's surface area (REDARS2 Technical manual 2006, p.G-27). However, this deck area is in square meters and is multiplied by a \$ cost per square feet which underestimates the direct damage cost by almost 10 times.

The assumptions used in the analyses have been reviewed and Oregon specific data, obtained from consultation with ODOT, are incorporated in the model. The replacement costs (REP_2) are calculated as a product of a base cost of \$165/ft^2, the deck area and a factor of 3.2 (to incorporate associated costs such as approaches, traffic control, etc.) with a \$3 million minimum cost. And when estimating the cost of a new bridge to replace an old bridge, a further multiplication factor of 1.2 is used, because the new bridge is expected to be of a larger dimension than the old one (ODOT 2009). The repair cost is computed as the product of a repair cost which depends on the bridge's damage state, and replacement cost (Table 6).

Replacement Cost (REP_2) = max of

- \$165/ft^2 x the deck area x 3.2 x 1.2 (when using a "old" bridge to estimate the cost of replacement of a "new" bridge)
- \$3 million

Table 6: Average Repair Cost Estimate

Damage State	Default	User- Defined (Oregon specific)	
	Cost of Repair (Replacement Cost = Bridge Deck Surface Area x \$150/ft^2)	**Cost of Repair**	**Minimum Cost**
Slight	0.03 x Replacement Cost	0.03 x Replacement Cost (REP_2)	\$ 100, 000
Moderate	0.25 x Replacement Cost	0.25 x Replacement Cost (REP_2)	\$ 750, 000
Extensive	0.75 x Replacement Cost	1.0 x Replacement Cost (REP_2)	\$ 3,000, 000
Complete	1.0 x Replacement Cost	1.0 x Replacement Cost (REP_2)	\$ 3,000, 000

3.2 ROADWAY TRANSPORTATION NETWORK DATABASES AND TRANSPORTATION DATA

The REDARS2™ Import Wizard uses nationally available FHWA datasets to enable prompt creation of REDARS2™ study regions. Gathering the various databases required by the program’s Import Wizard in their necessary formats and getting the Wizard to successfully process the data were significant steps in creating the initialization process.

National Highway Planning Network (NHPN) and Highway Performance Monitoring System (HPMS) are nationally available transportation databases that model the spatial configuration and attributes of the roadways in the study area. The databases are assembled by the individual states and distributed by FHWA.

Roadway systems are divided into a set of sub regions called Traffic Analysis Zones (TAZs) to monitor user trip demands on the roadway system. This subdivision is done by the local and state governments. TAZs are small areas approximately the size of a census tract. Origin-Destination (O-D) data estimates the location of travel origins and destinations and the corresponding number of trips from and to all the different TAZs in the region, and is compiled by local metropolitan planning organizations from periodic public surveys. TAZ and O-D data provide REDARS2 with a means to calculate travel time and demand between the different sub-regions of the state as well as value or economic loss when a particular route is shut down. The transportation data was located in the previous study done by PSU (Dusicka 2008).

A refined the State Wide Traffic Data of the recently completed statewide network and Traffic Analysis Zone (TAZ) system that more closely represents the State has been acquired from Oregon Department of Transportation’s Transportation Planning and Analysis Unit. The old National Highway Planning Network (NHPN) which has 2473 links covering interstates, major US and state highways and urban arterials (Figure 5) and the refined network, which is adjusted for roads only, includes nearly 54,000 links, including comprehensive statewide coverage and 5800+ segments out of state (Figure 6). And the old limited TAZ system has 146 zones, including 25 externals and 4 zones in Clark County, WA (Figure 7). The enhanced zone system includes about 2950 zones of which 2583 in Oregon and 377 in WA, ID, NV, and CA (Figure 8).

Using the REDARS2 Import Wizard has proved to be problematic in implementing the available refined transportation data. Data type incompatibility was the main initial issue. The Import Wizard requires the NHPN data in an uncompressed Arc Interchange file format called”e00”. NHPN and Highway Performance Monitoring System (HPMS) databases are no longer published in the format that REDARS2 accepts. Therefore, one attempt was to bypass the Import wizard and create the Microsoft Access Database (MDB) files that resemble NHPN and HPMS data that required by the REDARS2. This also has proven to be a hurdle given the lack of documentation provided and the interconnectivity of the data. The Import Wizard creates five MDB files and creating these five files which dependent on each other. After resolving a number of the data related issues, the size of the database has proven to be too large for REDARDS2 to handle. Consequently, the refined State Wide Traffic Data for Oregon was not successfully incorporated into the REDADRS2 model.

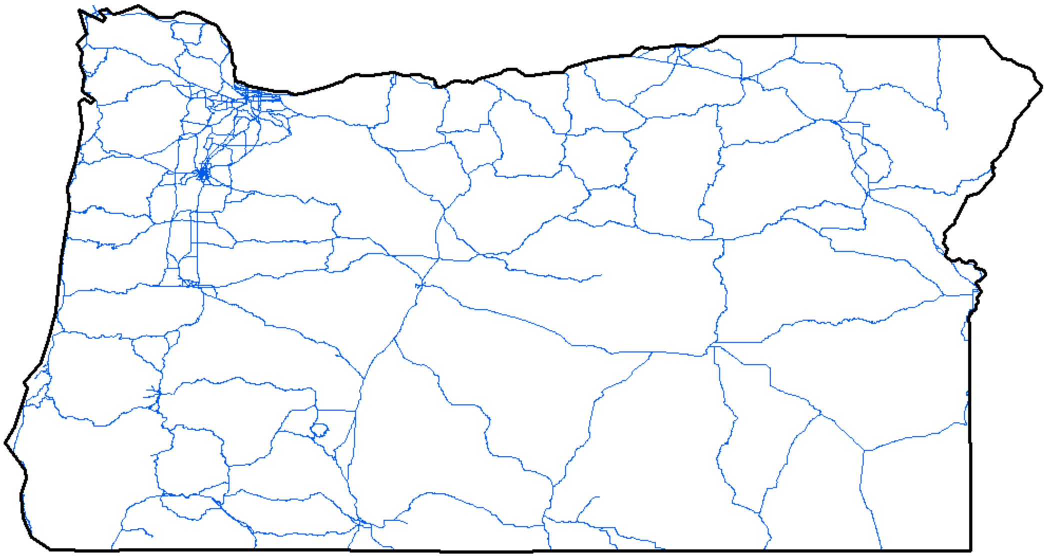

Figure 5: Old FHWA National Highway Planning Network.

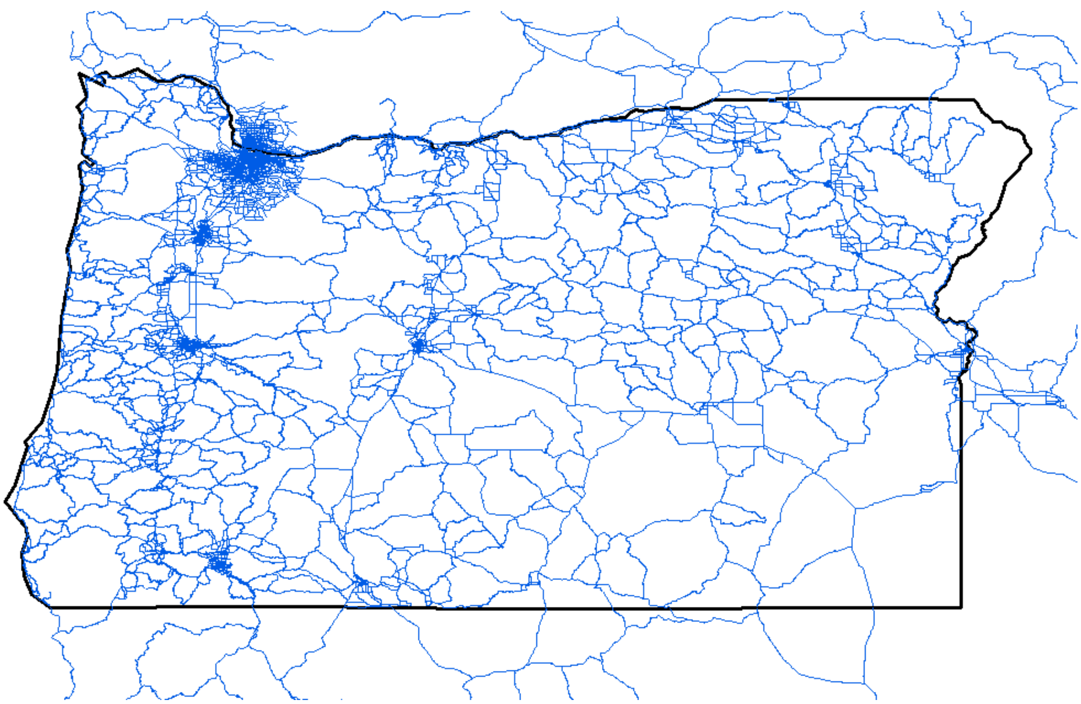

Figure 6: New Oregon DOT SWIM Network

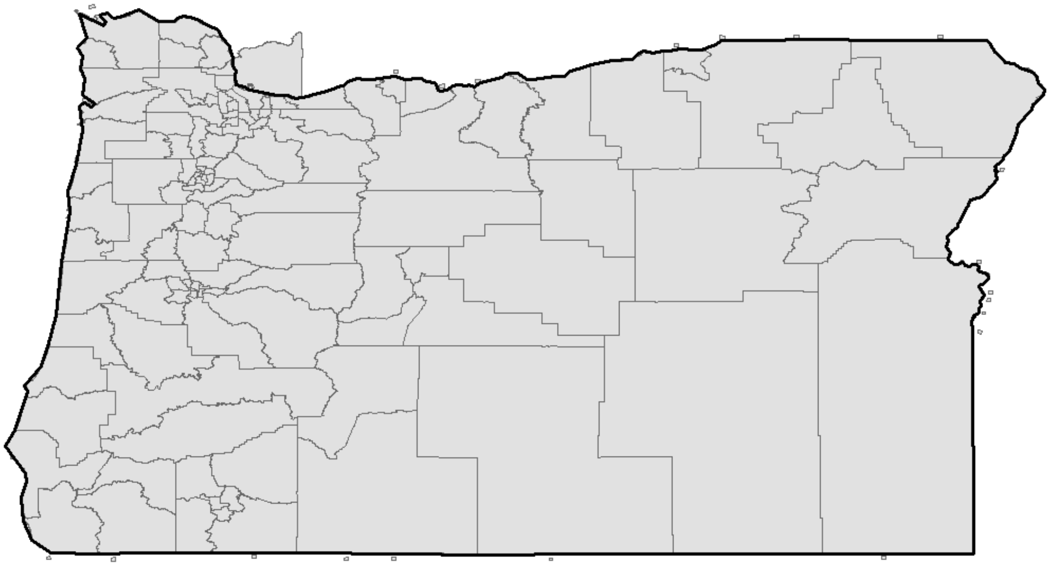

Figure 7: Old Limited TAZ System

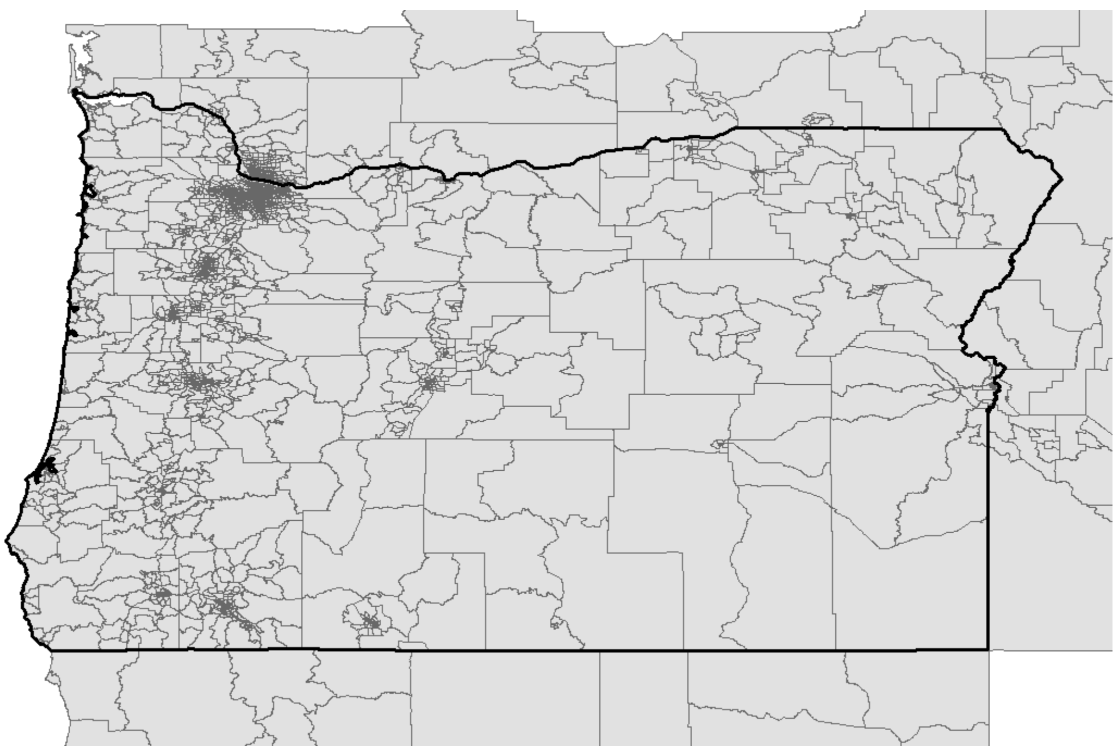

Figure 8: Comprehensive TAZ System

4.0 GEOTECHNICAL ASSESSMENT

4.1 LIQUEFACTION HAZARD ASSESSMENT

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for tremendous amounts of damage in historical earthquakes around the world.

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

4.1.1 Previous investigations

REDARS2 documents the capability of computing ground displacements due to liquefaction-induced vertical settlement and lateral spread. The first generation study conducted by PSU obtained data for both liquefaction hazard and shear wave amplification factors (NEHRP ratings) for the sites of ODOT bridges (Dusicka 2008). Large scale maps of liquefaction potential and of NEHRP ratings had to be created in the Oregon highway network model since there was no complete list of NEHRP ratings for bridges available for reference when assessing ODOT bridges. Bridges were assessed as potentially liquefiable or non-liquefiable. The initial approach to assessing liquefaction for the respective bridges located within the study region was to screen for liquefaction potential based on the geological sedimentary deposits, water table depth less than 15m (49ft), evaluation of sensitive clays, and final soil classification. However, since ODOT's BDS did not contain any of the relevant geological data to eliminate any of the structures based on geological composite, the soils profiles were initially only eliminated relative to the water table depth.

4.1.2 Liquefaction Assessment in REDARS2

The Tokimatsu-Seed (1987) model is used in the assessment of liquefaction induced vertical settlement (Figure 9). To apply this model, peak ground acceleration and soil data is input and identification of layers that could settle is required. The soil data that go into this are the corrected blow count numbers, layers depth below the ground surface, the thickness and the layers total overburden pressure and effective overburden pressure. Basic calculations in Tokimatsu-Seed model has the form of curves that define the combination of demand cyclic stress ratio and corrected blow counts that lead to various fixed values of volumetric strain.

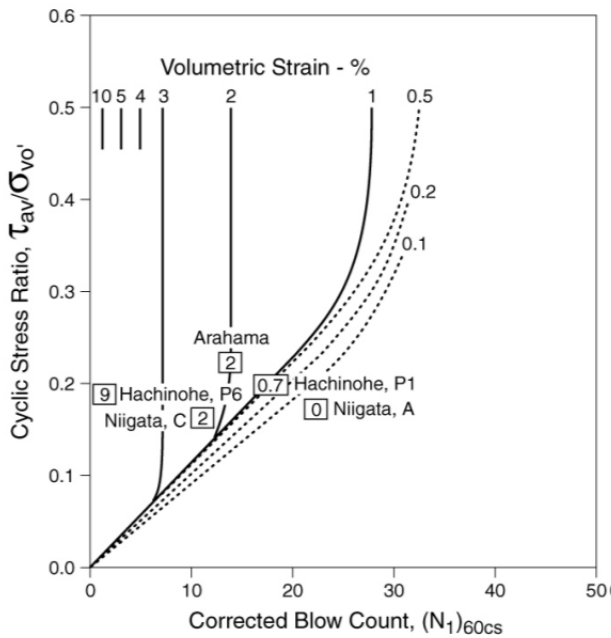

Figure 9: Liquefaction-Induced Volumetric Strains for Each Saturated Sand Layer in Site (Tokimatsu-Seed, 1987)

The demand cyclic stress ratio, CSR is computed as a function of peak ground acceleration (PGA), total overburden pressure, effective overburden pressure, gravity and a depth dependent stress reduction function. And with this cyclic stress ratio and the layer's corrected blow counts $(N_1)_{60}$, equations that represent the various Tokimatsu-Seed volumetric-strain curves are used to determine that layer's volumetric strain for a particular earthquake. Volumetric strain is then multiplied by its thickness in order to obtain the change in thickness of that layer. This is done for each saturated layer and the changes in thickness are added in order to obtain the total vertical settlement at the site (Werner et al. 2006).

Damage states in REDARS2 are determined from ground motion analysis or the liquefaction-induced peak ground displacement (PGD) analysis. In REDARS2, a 3.9" total settlement is defined as damage state 4 (Extensive damage) and a total settlement of 13.9" is defined as damage state 5 (Complete collapse). Settlements less than 3.9" are considered damage state 1 (No damage). Permanent ground displacement capacities for various bridge damage states before being modified for uncertainties are shown on APPENDIX B. Preliminary observations from the analysis seem to indicate a disproportionate influence in accounting for damage when liquefaction was being considered. Sensitivity analyses traced the issue to REDARS2 internal calculation of settlement, which cannot be altered without reprogramming and the source code is not easily understood due to lack of code comments and provided documentation. Figure 10 shows a sample analyses for damaged bridges for an earthquake scenario of magnitude 7.0 located near Scotts Mills. A bridge on US 101 over 200 miles away from the earthquake source which experienced a PGA of 0.013g showed a damage state 5 or complete collapse. However, preliminary results from a recent validation of this model based on observed damage states after the 1993 Scotts Mills Earthquake indicated unrealistic amounts of damage induced on bridges due to liquefaction. In that earthquake, no bridge collapsed due to liquefaction. Figure 11 shows the damaged bridges for an earthquake scenario of magnitude 5.6 at Scotts Mills. A bridge on US 101 which sees a PGA of 0.006g shows a damage state 5 or complete collapse.

The sensitivity of the model for varying PGA only starts at a very low acceleration, which was found to be approximately 0.00045g. For values larger than that, it seems to only depend on the blow count number and thickness of layer. Following are graphs that show the sensitivity of the model to varying soil properties and PGA (Figure 12). Hence, REDARS2 cannot be utilized for liquefaction induced seismic analysis without addressing the internal computation algorithms. Therefore, it is necessary to filter out bridges which are potentially liquefiable and do further liquefaction analysis of those bridges independent of REDARS2.

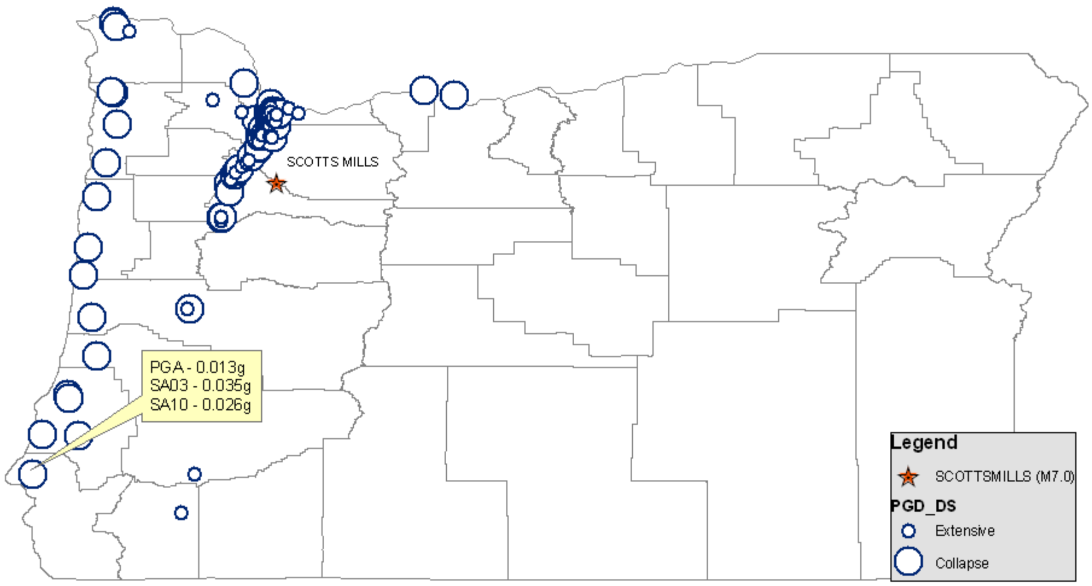

Figure 10: Liquefaction induced damages on bridges for a scenario earthquake at Scotts Mills (M7.0).

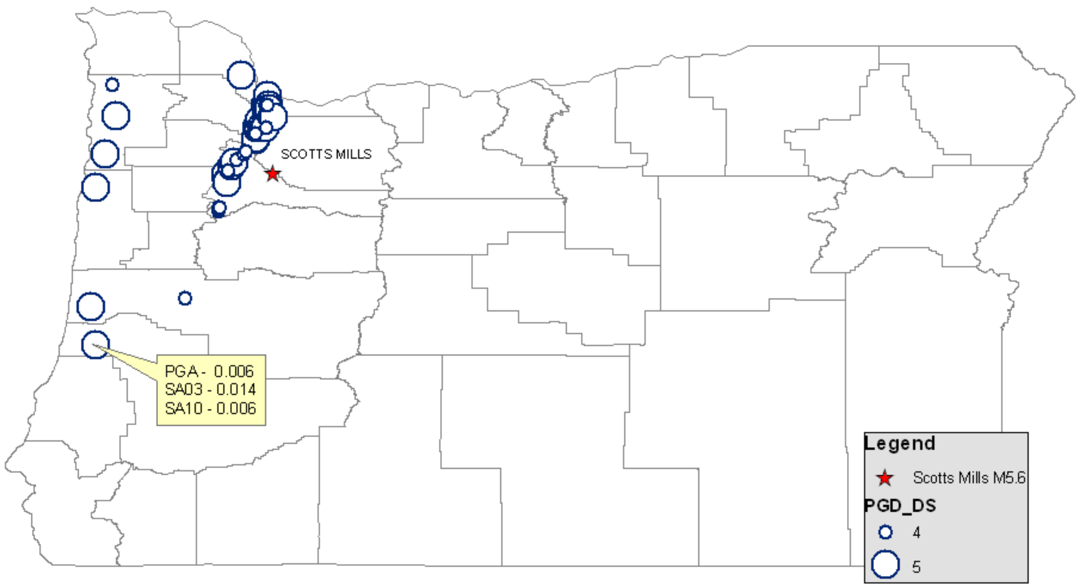

Figure 11: Liquefaction induced damages on bridges for a scenario earthquake at Scotts Mills (M5.6)

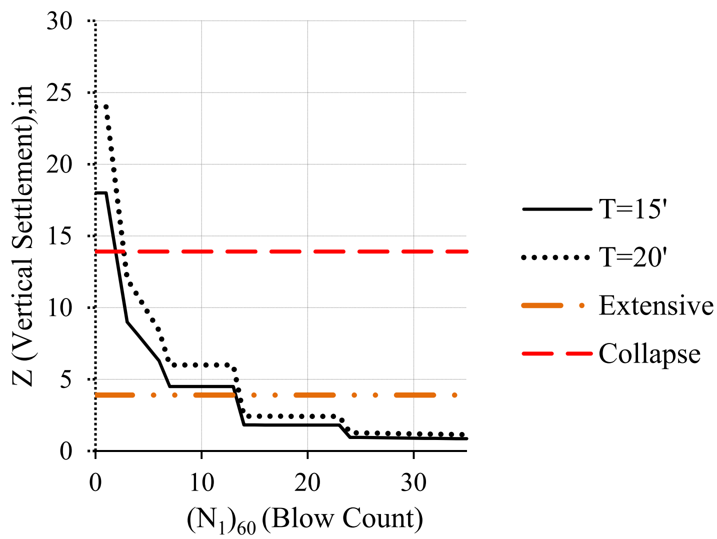

Figure 12: Vertical Settlement vs. (N1)60 for different thickness (T) of layers at PGA ≥0.00045

4.1.3 Liquefaction Potential Assessment Independent of REDARS2

The initial step of the liquefaction hazard evaluation is to characterize the relative liquefaction susceptibility of the site based on the geological sedimentary deposits, water table depth and evaluation of sensitive clays, and final soil classification. Soil survey data which are a product of the National Cooperative Soil Survey, from the USDA website were downloaded and used for initial screening for liquefaction potential. These soil data maps include water table depth and engineering soil properties such as soil classification according to unified soil classification (UCS) and AASHTO.

4.1.4 Methodology and Analysis

Oregon has 39 soil survey areas and each survey area has been mapped at different scales and different levels of detail. Therefore, the map unit symbols, soil properties and interpretations do not completely agree across the soil survey boundaries. The soil data maps were then spatially joined with the bridge layer. The interpretation of the map unit symbols was then extracted from the soil data map.

These soil data (SSURGO data – **S**oil **Sur**vey **Geo**graphic) are downloaded via the Soil Data Mart (http://soildatamart.nrcs.usda.gov) and an ArcGIS extension, Soil Data Viewer, is used to develop the shape files. Once the soil map is in place, the soil data maps were then spatially joined with the bridge layer. The interpretation of the map unit symbols was then extracted from the soil data map and the bridges were sorted according to the map unit symbols and filtered out according to their liquefaction potential.

4.1.5 Results of Liquefaction Potential Assessment

The bridges in the assessment are 1938 and of those almost 500 are flagged as "Can liquefy." Liquefaction hazard assessment of these bridges should be done outside of REDARS2 since REDARS2 calculation of Settlement is not reliable (Figure 13).

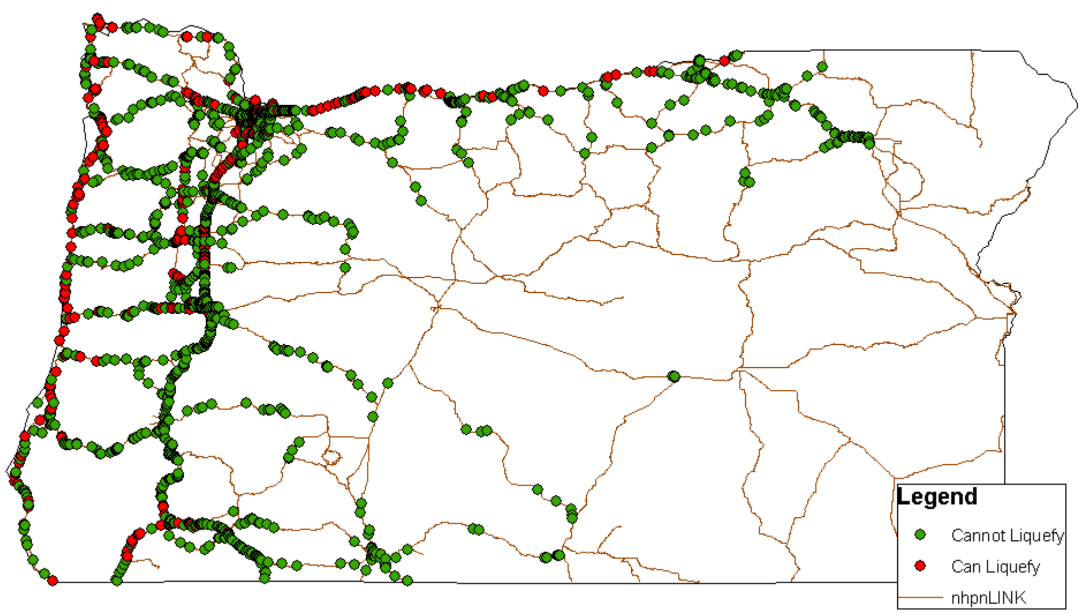

Figure 13: Initial Liquefaction probability screening of bridges.

5.0 ANALYSIS OF OREGON MODEL

5.1 EARTHQUAKE MODEL

5.1.1 REDARS2 Earthquake Model

REDARS2 uses the ground motion model by Abrahamson & Silva (1997) for crustal earthquakes in the western United States. Ground motion is expressed as a function of the earthquake magnitude, distance from site, soil conditions, and type of faulting, whether the site is along the hanging wall of footwall of the ruptured fault plane, and inter-event and intra-event uncertainties.

In REDARS2 the Seismic Risk Analysis (SRA) methodology is a synthesis of models developed by earth scientists, geotechnical and structural earthquake engineers, transportation engineers and planners, and economists. The methodology can develop multiple types/forms of results from deterministic or probabilistic approaches and from local to large geographic areas. Such results can be developed for use in pre-earthquake assessment of various options for seismic risk reduction after an actual earthquake (Werner et al., 2006).

REDARS2 typically utilizes publicly available databases to define roadway topology and attributes, bridge locations and attributes, origin-destination (O-D) zones and pre-earthquake trip tables and site-specific NEHRP soil conditions. REDARS2 has an integrated ability to analyze the transportation network as a system, considering both direct losses due to damage and indirect losses due to traffic flow disruption.

The methodology to carry out deterministic or probabilistic seismic risk analysis is depicted in Figure 14. For probabilistic SRA, results are developed for multiple simulations, in which a "simulation" is defined as a complete set of system SRA results for one particular set of randomly selected input parameters and model parameters. The model and input parameters for one simulation may differ from those for other simulations because of random and systematic uncertainties. For deterministic SRA, one set of results is developed either for median input and model parameters or for one set of randomly selected parameters. This multi-disciplinary procedure uses geoseismic, geotechnical and structural engineering, repair / construction, transportation network, and economic models to estimate hazards, component performance, system performance and losses such as economic impacts due to repair costs and losses due to travel time delays.

5.1.1.1 Subduction Zone Analysis

Subduction events are a significant component of the earthquake risk in the Pacific Northwest and Northern California. A major limitation of REDARS2 was encountered in trying to assess the impact of subduction zone earthquakes on the transportation network because only shallow crustal analysis is available (Abrahamson & Silva 1997). The REDARS2 program however did not include the ability to consider these types of earthquakes and therefore considerably hampers the usefulness of the analyses (Cho et al 2006). Therefore to incorporate that capability, instead of modifying the program, shakemaps generated by USGS for Cascadia subduction zone scenario events have been used as ground shaking source. USGS developed shakemaps for a subduction zone event of M9.0 (Figure 15), M8.3 North (Figure 16) and M8.3 South (Figure 17).

For shakemaps, ground motions are estimated using an empirical attenuation relationship, which is a predictive relationship that allows the estimation of the peak ground motions at a given distance and for an assumed magnitude. And these shakemaps can be used as ground shaking source in REDARS2. Below are PGA maps generated by USGS for a subduction zone scenario event.

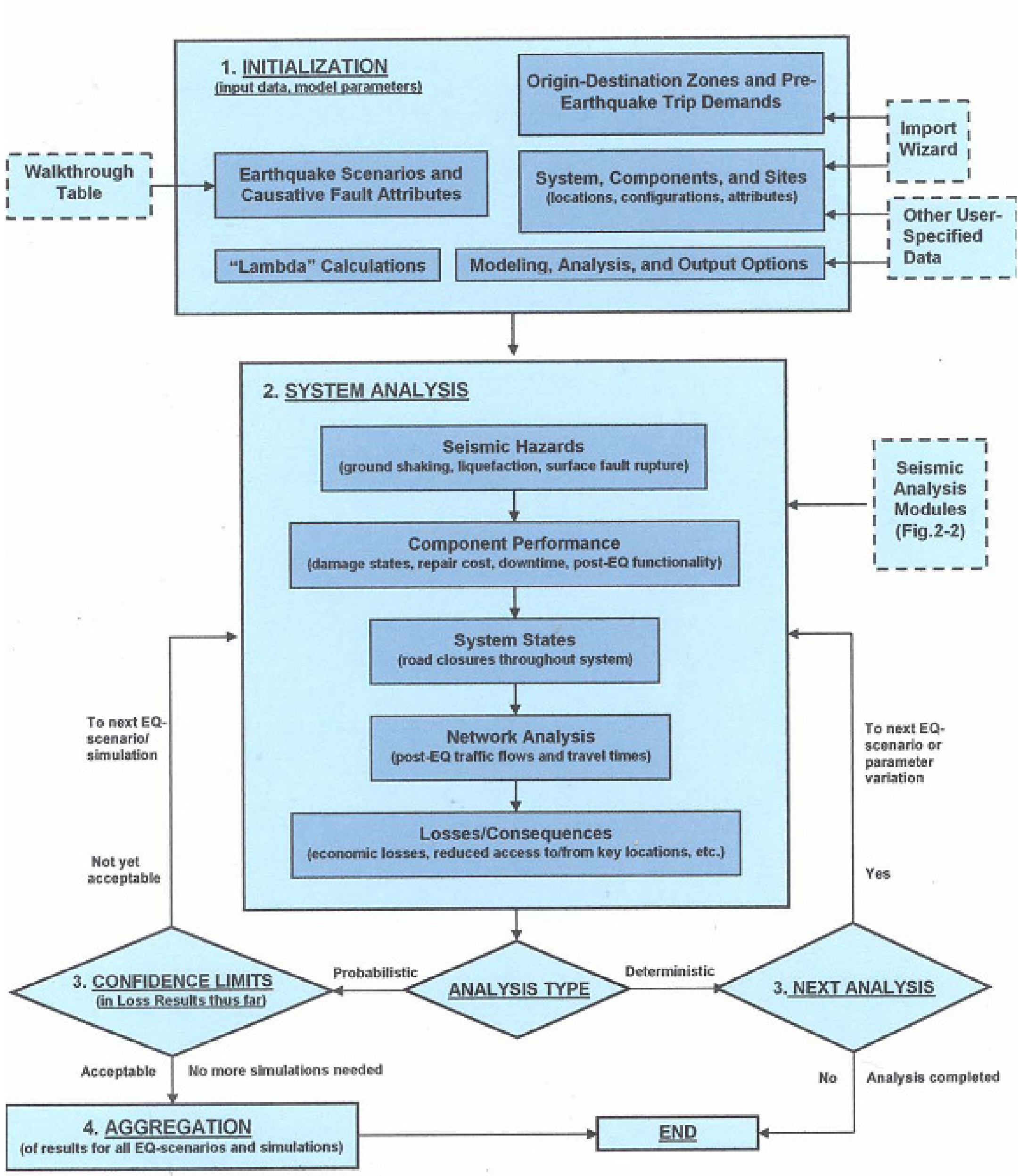

Figure 14: Seismic Risk Analysis of Roadway Systems
(TECHNICAL MANUAL: REDARS2 2 METHODOLOGY AND SOFTWARE FOR SEISMIC RISK ANALYSIS OF HIGHWAY SYSTEMS)

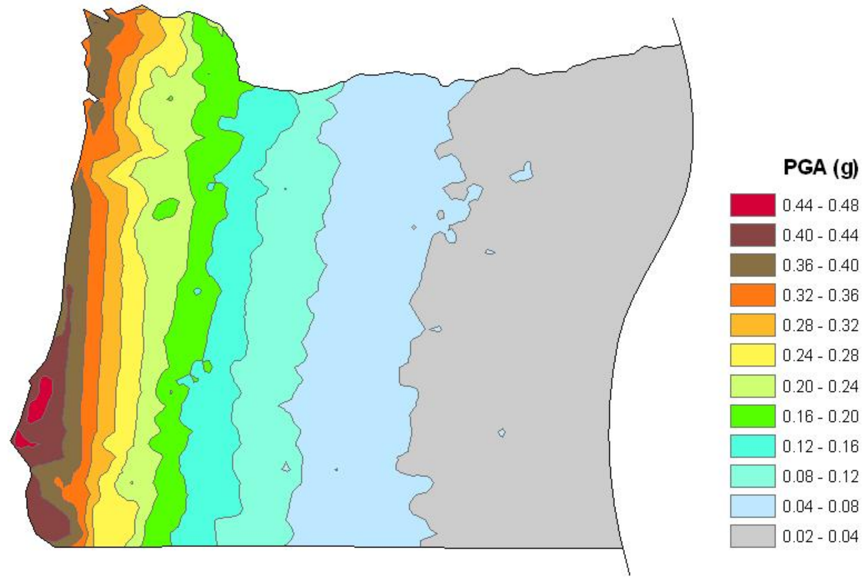

Figure 15: Scenario ShakeMap for CSZ M9.0.

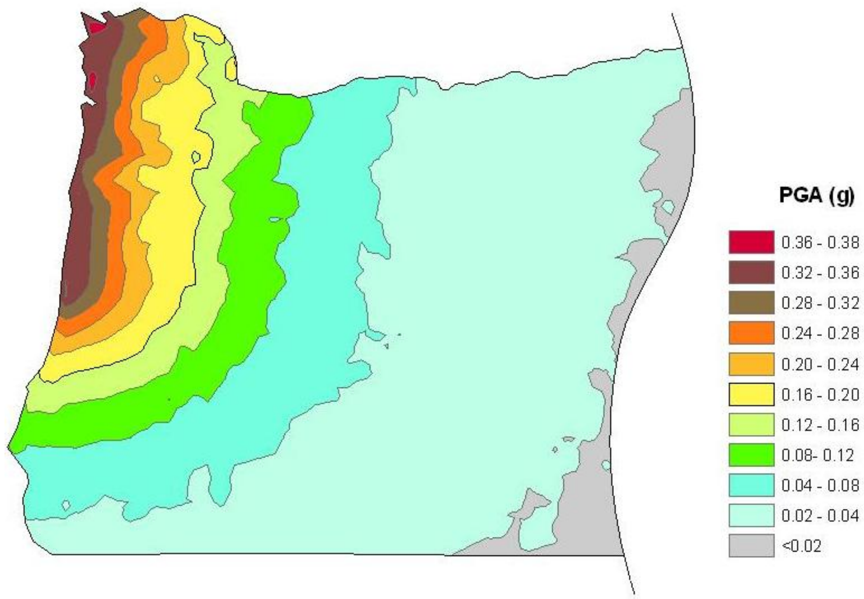

Figure 16: Scenario ShakeMap for CSZ North M8.3.

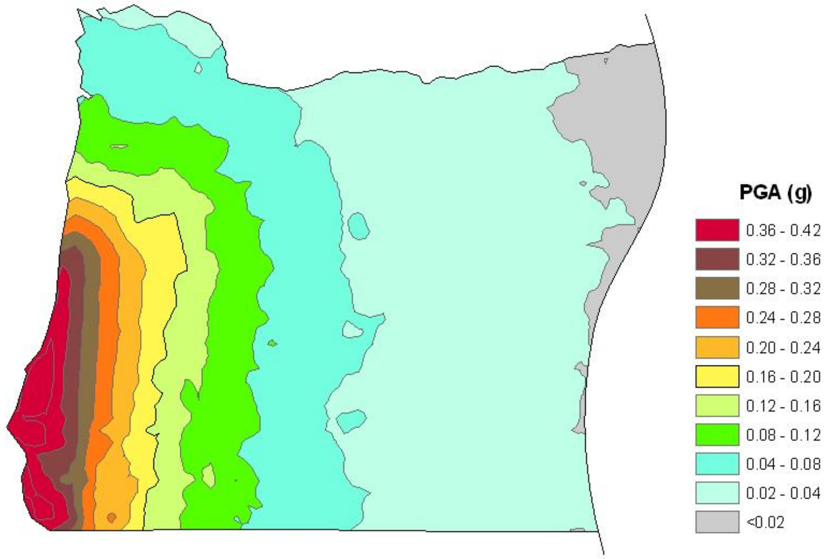

Figure 17: Scenario ShakeMap for CSZ South M8.3.

5.1.2 HAZUS-MH Earthquake Model

The HAZUS-MH methodology incorporates available state-of-the-art models in the earthquake loss estimation methodology. These modules include damage loss, such as induced damage due to fire following earthquake and indirect economic loss. A nationally applicable scheme is developed for classifying components. However, unlike REDARS2 interdependence of components on overall system functionality is not addressed by HAZUS. Hence, a network system analysis would need to be performed separately following the damage state analyses (FEMA 2003).

5.2 COMPONENT DAMAGE ANALYSIS

5.2.1 Bridge Damage State model

In REDARS2, bridge damage resulting from an earthquake event is classified into damage states ranging from no damage to complete collapse. The bridge model utilized for SRA of the Oregon transportation network was based on HAZUS99-SR2, which defines bridge capacities in terms of spectral accelerations leading to the onset of five damage states listed in Table 7 for each of the several "standard bridge" classifications.

Table 7: Damage States considered in REDARS2 Bridge Model

Damage State Designation		Description of Typical Expected Damage
Number	Level	
1	None	Up to first yield.
2	Slight	Minor cracking and spalling of the abutment, cracks in shear keys at abutment, minor spalling and cracking at hinges, minor spalling of column requiring no more than cosmetic repair, or minor cracking of deck.
3	Moderate	Any column experiencing moderate shear cracking and spalling (with columns still structurally sound), moderate movement of abutment (< 5.1 cm) (< 2 inches), extensive cracking and spalling of shear keys, connection with cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure, or moderate settlement of approach.
4	Extensive	Any column degrading without collapse (e.g., shear failure) but with column structurally unsafe, significant residual movement of connections, major settlement of approach fills vertical offset or shear key failure at abutments, or differential settlement.
5	Complete	Collapse of any column or unseating of deck spans leading to collapse of deck. Tilting of substructure due to foundation failure.

Once the spectral acceleration capacity for a given bridge is estimated, a ground motion model is used to estimate the bridge's site-specific demand ground motions (in terms of spectral accelerations S_a (1.0) and S_a (0.3)) for each scenario earthquake. The capacity for the bridges is computed including effects of uncertainties. However, the capacity modification factors are developed by statistical analysis for each damage state and are the mean values.

Estimation of ground motions for different scenario earthquakes and simulations includes effects of uncertainties in earthquake magnitude and location, ground motion attenuation characteristics, and soil amplification effects. The Abrahamson-Silva (1997) ground motion model, that is the attenuation in REDARS2, estimates spectral accelerations caused by shallow crustal earthquakes in active tectonic regions of the Western United States, excluding subduction earthquakes. The Abrahamson-Silva ground motion model expresses the natural logarithm of the ground motion as a function of the earthquake magnitude, source-site distance, local soil conditions, and type of faulting, whether the site is along the hanging wall or footwall of the ruptured fault plane, and inter-event and intra-event uncertainties. This functionality is represented through a series of numerical coefficients that are used to compute each term in this equation. Once the bridge's demand spectral acceleration is computed for a given scenario earthquake, it is compared to each bridge's spectral acceleration capacity that leads to the onset of each damage state in order to estimate the bridge's damage state for the particular earthquake and simulation. Table B. 1gives the median ground motions leading to onset of various damage states for "Standard" Bridges.

5.3 TRANSPORTATION NETWORK ANALYSIS

REDARS2 transportation network analysis of the systems takes into account the spatial distribution of the system and account for the redundancy in the system or lack thereof. One of the procedures of network-analysis was based on Variable-Demand Model (VDM). VDM accounts for a reduction in trip rate and an increase in travel time according to the post-earthquake changes in network capacity. The difference in system cost caused by congestion is accounted for. The difference in trip rate is also considered as another type of social cost, together with the value of foregone trips (Werner 2006).

5.4 ECONOMIC LOSS CALCULATION IN REDARS2

5.4.1 Repair Cost

Estimation of economic impacts of earthquake damage to the system is one of the most important end results from SRA of roadway systems. Bridge damage results not only in high cost of structural repair but also safety concerns by severely disrupting traffic flow which in turn will impact post-earthquake emergency response, repair and reconstruction operations and long term economic consequences due to the valued loss of time when commuter and freight travel slows down due to the disrupted network. From this, it is apparent that earthquake damage to certain components (e.g., those along important and non-redundant links within the system) will have a greater impact on the system performance than will other components.

The SRA methodology uses the bridge and network data to estimate direct and indirect economic losses due to disruption in the system. The SRA considers repair costs, losses due to earthquake-induced travel-time delays and losses from trips foregone due to earthquake-induced increases in traffic congestion. The default REDARS2 model estimates cost as the product of a unit replacement cost (REP_1) assumed to be \$150/ft^2 and the bridge deck's surface area. However, these repair costs ratios and unit replacement costs can be overridden by the user. Hence, the replacement in this study are calculated as a product of a base cost of \$165/ft2, the deck area and a factor of 3.2 with a \$3 million minimum cost. And when estimating the cost of a new bridge with an old bridge, a further multiplication factor of 1.2 is used (Equation 1), because the new bridge is expected to be of a larger dimension than the old one. The repair cost is computed as the product of a repair cost ratio which depends on the bridge's damage state, and the replacement cost (REP_2) (Table 6).

Equation 1

Replacement Cost = max of

- \$165/ft ^{2} x the deck area x 3.2 x 1.2 (when using a "old" bridge to estimate the cost of replacement of a "new" bridge)
- \$3 million

Table 8: Average Repair Cost Estimate

Damage State	Default	User- Defined	
	Cost of Repair (Replacement Cost = Bridge Deck Surface Area x $\$150/ft^2$)	**Cost of Repair**	**Minimum Cost**
Slight	0.03 x Replacement Cost (REP_1)	0.03 x Replacement Cost (REP_2)	\$ 100, 000
Moderate	0.25 x Replacement Cost (REP_1)	0.25 x Replacement Cost (REP_2)	\$ 750, 000
Extensive	0.75 x Replacement Cost (REP_1)	1.0 x Replacement Cost (REP_2)	\$ 3,000, 000
Complete	1.0 x Replacement Cost (REP_1)	1.0 x Replacement Cost (REP_2)	\$ 3,000, 000

5.4.2 Losses due to Travel-Time Delays and Trips Foregone

REDARS2™ methodology for calculating the social cost of earthquake-induced traffic disruption using zone-to-zone trip demands and the corresponding change in travel time. This social cost includes the value of time due to increased traveler time on the roadway and the value of trips foregone. TAZ and O-D data provide REDARS2 with a means to calculate travel time and demand between the different sub-regions of the state as well as value or economic loss when a particular route is shut down. The unit cost and density parameters of each vehicle class are then input in REDARS2. These parameters are: a) the value of one hour of travel time (VOT); and b) the number of passenger cars equivalent to one Origin-Destination (O-D) unit (PCU) for each class of O-D data represented. VOT parameters are used to calculate economic impact based on loss of travel time after an earthquake; PCU parameters are used to calculate highway densities and travel times.

Oregon representative data that can be used to estimate VOT parameters for Auto, Light Truck and Heavy Truck vehicle classes were supplied by Dr. Chris Monsere of PSU Civil Engineering, and individuals at ODOT's Transportation Planning Analysis Unit (TPAU). This data was based on a calculation weighing several value categories including Oregon average wages and fringe benefits, costs of employees, freight inventory values and average vehicle occupancies. The resulting weighted average VOTs for each vehicle class were: a) Auto: \$15.31/hr; b) Light Trucks: \$19.53/hr; c) Heavy Trucks: \$30.43/hr. Since O-D data only represented trips for one "Truck" class, its associated VOT was interpolated between the values for Light Truck and Heavy Truck vehicle classes considering a traffic composition of 25% light trucks and 10% heavy trucks. This resulted in a Truck VOT value of \$22.69/hr. REDARS2's default PCU values for California were 1.0 for the Auto class and 2.5 for the Freight class which were considered reasonable values for Oregon (Dusicka 2008).

The default traffic model assumes a bridge is either fully closed or fully open to traffic after damage has occurred and during repair. Table 9 indicates the variation in times to reach fully open status for the different damage states. The model also accounts for the effect damage to a bridge has on the traffic flow of any underlying roadway; these disruption times are the same as above for each damage state except in the event of complete damage, in which case the underlying roadway is assumed to be fully open to traffic 30 days after the event. Any of the default values determining traffic states can be modified by the user, including the default assumption that a bridge is either fully open or fully closed during repair. The user can override this assumption so that a "partially opened" bridge is considered where the number of lanes closed to traffic is a function of the damage state, total number of lanes and the number of bridge spans. Initial analyses conducted on the Oregon highway network model used the default values.

Table 9: Default Traffic States during Repair of Bridge Damage from Ground Motions (Werner 2006)

Bridge Damage State	**Number of Bridge Spans**	**Post-Earthquake Traffic State: Bridge**		**Post-Earthquake (EQ) Traffic State: Underlying Roadway**	
		Time after EQ, days	**Percent of Pre-EQ Traffic-Carrying Capacity**	**Time after EQ, days**	**Percent of Pre-EQ Traffic-Carrying Capacity**
None or Slight	-	0 days	100 %	0 days	100 %
Moderate	-	0-4 days > 4 days	0 % 100 %	0-4 days > 4 days	0 % 100 %
Extensive	-	0-12 days > 12 days	0 % 100 %	0-12 days > 12 days	0 % 100 %
Collapse	3 Spans	0-140 days > 140 days	0 % 100 %	0-30 days > 30 days	0 % 100 %
	4 Spans	0-180 days > 180 days	0 % 100 %	0-30 days > 30 days	0 % 100 %
	≥5 Spans	0-220 days > 220 days	0 % 100 %	0-30 days > 30 days	0 % 100 %

5.5 COMPARISON OF DAMAGE STATE ANALYSIS METHODS

REDARS2 and HAZUS-MH use the same Damage Functions for Bridges. Both use the model that makes use of damage functions developed by Basoz and Mander (1999) for estimating damage state of bridges. However, discrepancies were found in the analysis results.

Some of the discrepancies could be due to differences in the definition of bridge capacity between the two methods. For example, in REDARS2, bridge structures with Single-Column Bents (NBI class 205-206) and Concrete Box-Girder Superstructures (NBI class 605-606) do not have fragility curve defined (Table B. 2 and Table B. 3), hence, these bridge types will have their capacity (a median ground motion leading to onset of damage) of an unclassified bridge . Unclassified bridges in REDARS2 have fragilities defined as 0.80, 1.00, 1.20 and 1.70 as their median ground motions leading to onset of damage states; slight, moderate, extensive and collapse respectively for both conventionally and seismically designed bridges. However, in HAZUS-MH these bridges have fragility values that are classified. (Table B. 4). Bridges from the NBI class 205, 206, 605 and 606, the median ground motions leading to onset of damage states slight, moderate, extensive and collapse are 0.60, 0.90, 1.10, 1.50 for conventionally designed bridges and 0.90, 0.90, 1.10 and 1.50 respectively for seismically designed bridges. Similarly, steel truss-thru (NBI class 309) and steel truss-deck bridges (NBI class 310) are classified as "simply-supported steel superstructure bridges in REDARS2 and assigned median ground motions leading to onset of damage states slight, moderate, extensive and collapse of 0.25, 0.35, 0.45 and 0.70, whereas, in HAZUS-MH these bridges are not classified and are given fragility values of unclassified bridges (HWB28) of 0.80, 1.00, 1.20 and 1.70. For analyses cases using shakemaps as ground shaking sources, such as, Cascadia subduction event scenarios, Portland hills scenario & Klamath Falls scenario, the analysis results from REDARS2 showed that some bridges exhibited damage states higher than that of HAZUS-MH. And difference in capacity definition may have played a part.

One source of discrepancy can be due to some variations in the estimation of demand parameters. When using a point ground shaking source (e.g. Scotts Mills scenario), some bridges showed a higher damage state in HAZUS. In REDARS2, the Abrahamson-Silva (1997) ground motion model is adapted. And it estimates spectral accelerations caused by shallow crustal earthquakes in active tectonic regions of the Western United States, excluding subduction earthquakes. In HAZUS there is the option of selecting different attenuation models one of them being Abrahamson-Silva (1997) attenuation model. And one also needs to define parameters of the scenario event such as the epicenter location, moment magnitude, depth, width, orientation and dip angle of fault. The minor differences in damage states in this case could hence be partly attributed to some difference that could exist in parameter definitions. Therefore, definition of these parameters to be more or less the same will make the analysis results to be very comparable.

6.0 EARTHQUAKE SCENARIOS USED IN ANALYSIS

6.1 INTRODUCTION

The earthquake scenarios considered for this study are subduction zone earthquakes and crustal earthquakes. The Cascadia Subduction Zone (CSZ) during Oregon's short 150-year historical record, numerous studies have found widespread evidence that very large earthquakes have occurred, most recently about 300 years ago, in January 1700 (e.g., Atwater, 1987; Yamaguchi and others, 1997). The best available evidence and observations indicate that these earthquakes occur on average about every 500 years. Hence, it is important to make an analysis of a scenario CSZ earthquake so as to make a reasonable prediction of the effects of the assumed earthquake. This knowledge of potential damage will allow for planning and preparedness purposes.

Crustal earthquakes occur in the North American plate at relatively shallow depths of 10–20 km (6–12 mi) below the surface. The 1993 magnitude 5.6 earthquake at Scotts Mills, Oregon (Madin et al., 1993) and the 1993 magnitude 5.9 and 6.0 Klamath Falls, Oregon, main shocks (Wiley et al., 1993) are examples of crustal earthquakes that have occurred in Oregon. Consequently, crustal earthquake scenarios at Scotts Mills, Klamath Falls and Portland Metro area are examined for the Oregon Model.

6.1.1 Crustal Earthquake Scenarios in the Portland Metro Area

For an earthquake scenario of magnitude 7 at the Portland Metro Area, there were no complete collapses, 24 extensive, 56 major and 49 slight damage states. The losses calculated were $1,573 million for bridge repair and replacement and $68 million travel time related losses. Table 10 gives a breakdown of the number of damages and cost incurred per route (p.41).

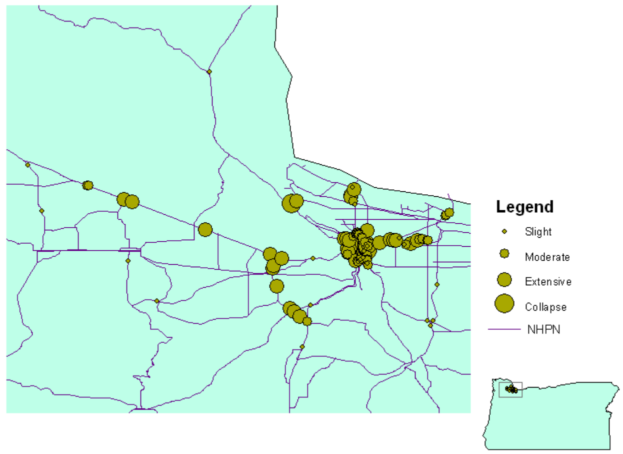

Figure 18: Component Damage States for a Magnitude 6.5 Scenario Earthquake around Portland Hills

6.1.2 Crustal Earthquake Scenarios in the Scotts Mills

For an earthquake scenario of magnitude 7 at Scotts Mills, there was one complete collapse, two extensive, two major and three slight damage states. The losses calculated were $14 million for bridge repair and replacement and $29 million in travel time related losses Figure 19. Table 11 gives a breakdown of the number of damages and cost incurred per route (p. 41)

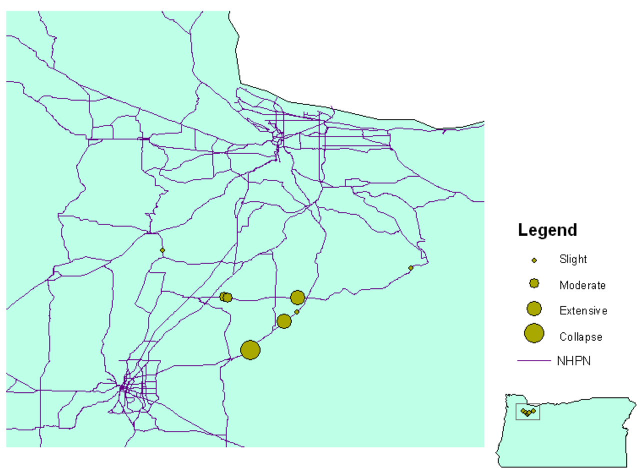

Figure 19: Component damage States for Magnitude 7.0 Scenario Earthquake around Scotts Mills.

6.1.3 Crustal Earthquake Scenarios in the Klamath Falls

A magnitude 6.5 scenario earthquake around Klamath Falls resulted in no complete collapses, 7 extensive, 6 moderate and 3 slight damage states. The losses $ 109 million for bridge repair and replacement and $3 million in travel time related losses (Figure 20).

Table 12 gives a breakdown of the number of damages and cost incurred per route (p.42).

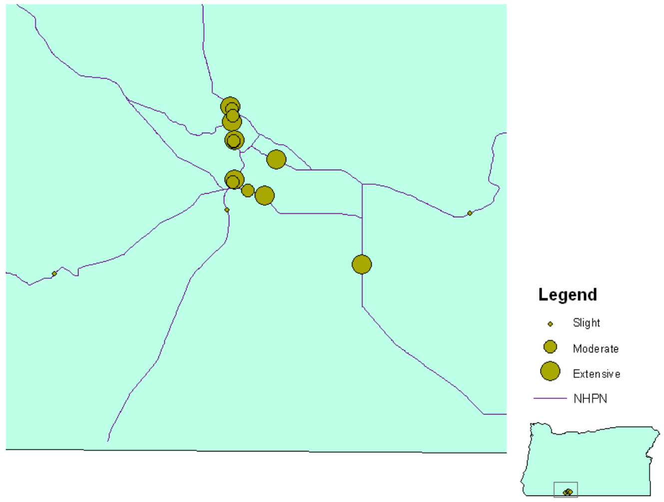

Figure 20: Component Damage States for Magnitude 7.0 Scenario Earthquake around Klamath Falls

6.1.4 Full Length Cascadia Subduction Zone Earthquake

The Abrahamson-Silva ground motion attenuation model only estimates spectral accelerations caused by shallow crustal earthquakes in active tectonic regions of the Western United States and excludes the subduction earthquakes. Therefore, for the CSZ earthquake events, a Cascadian Subduction Zone earthquake scenario ShakeMap is used as a ground shaking source.

An earthquake scenario of magnitude 9 at the Cascadian Subduction Zone resulted in 6 complete collapses, 64 extensive, 106 major and 164 slight damage states. The losses calculated were $1,080 million for bridge repair and replacement and $177 million travel time related losses. Figure 21 shows a map of the component damage states for the western part of Oregon. Table 13 gives a breakdown of the number of damages and cost incurred per route (p. 42).

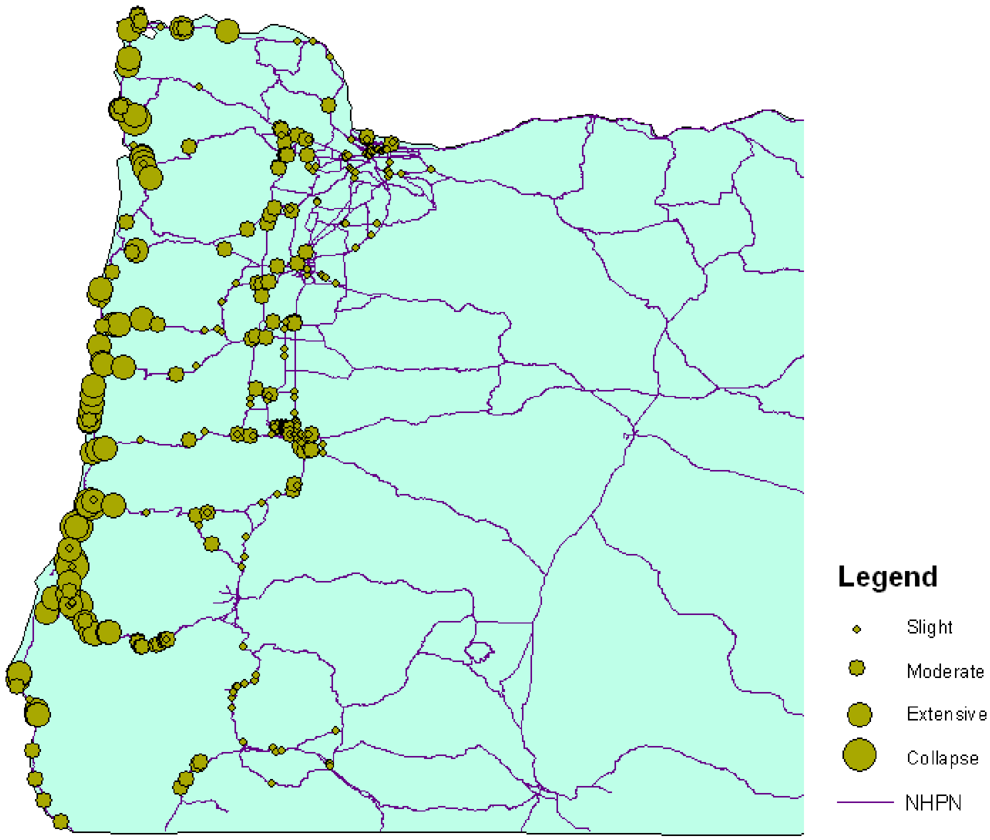

Figure 21: Component Damage States for a M9.0 Cascadia Subduction Zone Scenario earthquake.

6.1.5 Cascadia Subduction Zone Earthquake near Northern Oregon

An earthquake scenario of magnitude 8.3 at the Cascadian Subduction Zone near northern Oregon produced no complete collapse, 28 extensive, 32 major and 152 slight damage states. The losses evaluated were $336 million for bridge repair and replacement and $8 million travel time related losses. Figure 22 shows a map of the component damage states for the western part of Oregon. Table 14 gives a breakdown of the number of damages and cost incurred per route (p. 43)

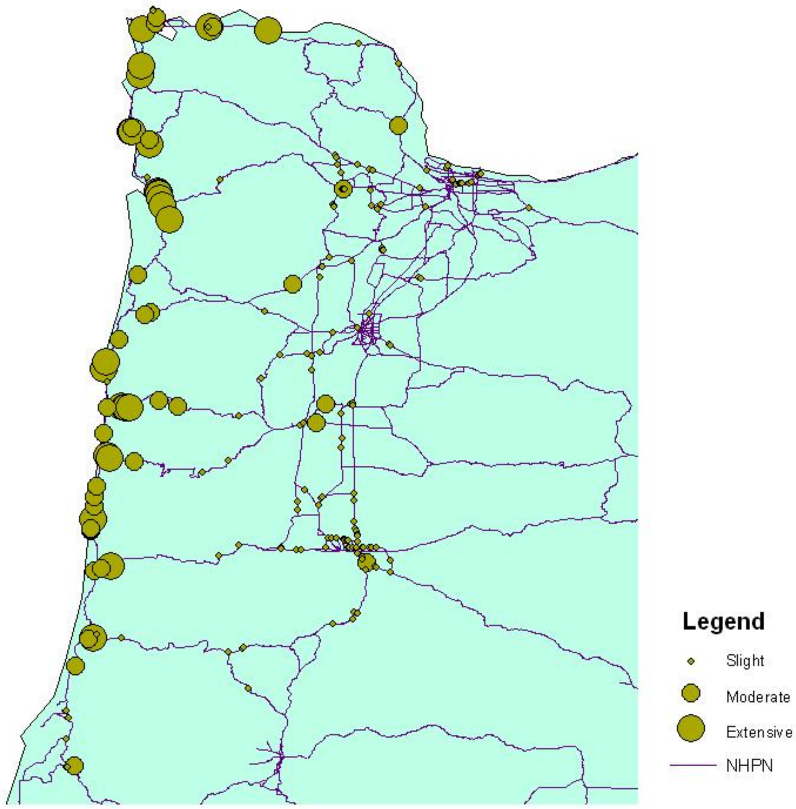

Figure 22: Component Damage States for a Magnitude 8.3 Cascadia Subduction Zone Scenario Earthquake near northern Oregon

6.1.6 Cascadia Subduction Zone Earthquake near Southern Oregon

An earthquake scenario of magnitude 8.3 at the Cascadian Subduction Zone near Southern Oregon produced 2 complete collapses, 23 extensive, 33 major and 123 slight damage states. The losses evaluated were $363 million for bridge repair and replacement and $94 million travel time related losses. Figure 23 shows a map of the component damage states for the Southwestern part of Oregon. Table 18 gives a breakdown of the number of damages and cost incurred per route (p. 50).

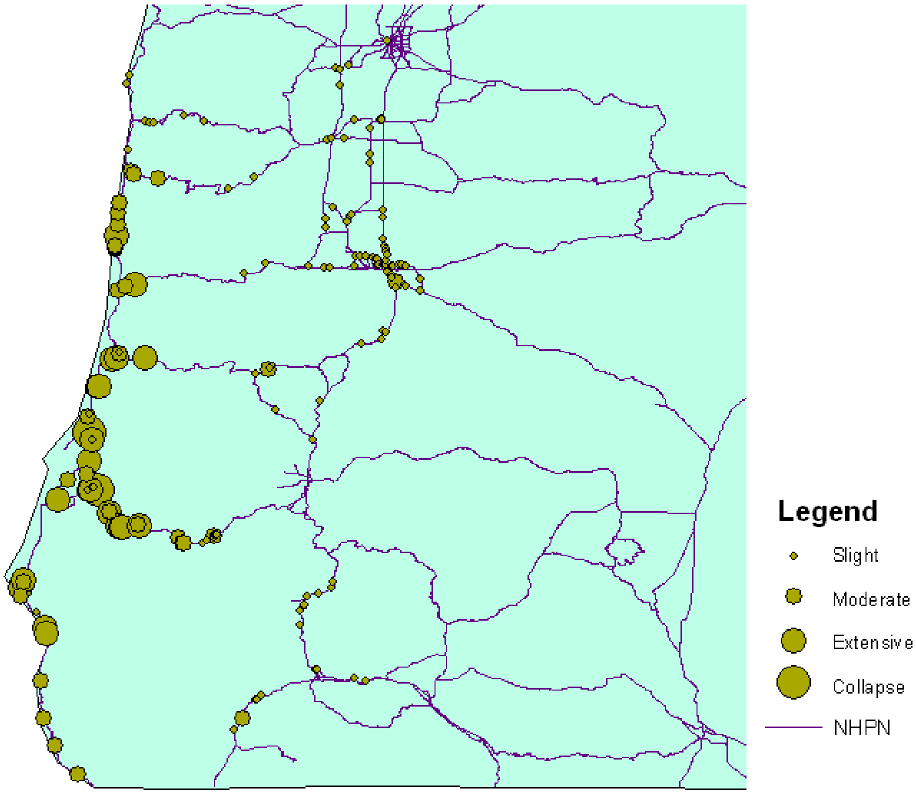

Figure 23: Component Damage States for a Magnitude 8.3 Cascadia Subduction Zone Scenario Earthquake near southern Oregon

6.1.7 Summary of number of damaged bridges per route

Table 10: Portland Hills M6.5 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5 (MWC)	8	12	10	1	$483	
I-5 (MLL)	0	0	0	0	0	
I-5 (DJJ)	0	0	0	0	0	
I-84	1	4	10	1	$143	
US-101	0	0	0	0	0	
US-26	3	3	7	0	$63	
I-205	5	4	0	0	$14	
I-405	3	12	9	2	$494	
US-30	1	0	1	1	$122	
US-20	0	0	0	0	0	
OR-38	0	0	0	0	0	
OR-42	0	0	0	0	0	
Others	6	6	11	0	$254	
Total	**27**	**41**	**48**	**5**	**$1,573**	**$68**

Table 11: Scotts Mills M7.0 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5	0	0	0	0	0	
I-84	0	0	0	0	0	
US-101	0	0	0	0	0	
US-26	0	0	0	0	0	
I-205	0	0	0	0	0	
I-405	0	0	0	0	0	
Others	3	2	2	1	$14	
Total	**3**	**2**	**2**	**1**	**$14**	**$29**

Table 12: Klamath Falls M7.0 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5	0	0	0	0	0	
I-84	0	0	0	0	0	
US-101	0	0	0	0	0	
US-26	0	0	0	0	0	
I-205	0	0	0	0	0	
I-405	0	0	0	0	0	
Others	3	6	7	0	$109	
Total	**3**	**6**	**7**	**0**	**$109**	**$3**

Table 13: CSZ M9.0 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5 (MWC)	4	1	0	0	$8	
I-5 (MLL)	16	3	1	0	$14	
I-5 (DJJ)	27	0	0	0	$5	
I-84	13	1	0	0	$10	
US-101	7	14	36	5	$685	
US-26	7	5	0	0	$9	
I-205	8	2	0	0	$10	
I-405	7	0	0	0	$2	
US-30	4	2	2	0	$26	
US-20	5	3	5	0	$19	
OR-38	3	2	1	0	$9	
OR-42	4	13	13	1	$158	
Others	59	60	6	0	$125	
Total	**164**	**106**	**64**	**6**	**$1,080**	**$177**

Table 14: CSZ North M8.3 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5 (MWC)	1	0	0	0	$0.4	
I-5 (MLL)	18	1	0	0	$5	
I-5 (DJJ)	3	0	0	0	$0.3	
I-84	8	0	0	0	$3	
US-101	7	18	19	0	$252	
US-26	9	0	0	0	$2	
I-205	4	0	0	0	$1	
I-405	0	0	0	0	$0	
US-30	3	2	2	0	$18	
US-20	2	2	4	0	$14	
OR-38	4	0	0	0	$1	
OR-42	4	1	0	0	$5	
Others	89	8	3	0	$35	
Total	**152**	**32**	**28**	**0**	**$337**	**$8**

Table 15: CSZ South M8.3 Scenario Analysis Results

Route	Damage States				Economic loss (in Million $)	
	Slight	Moderate	Extensive	Complete	Bridge Repair/Replacement	Travel Time Loss
I-5 (MWC)	0	0	0	0	0	
I-5 (MLL)	18	1	0	0	$5	
I-5 (DJJ)	20	0	0	0	$4	
I-84	0	0	0	0	0	
US-101	6	16	11	1	$207	
US-26	0	0	0	0	0	
I-205	0	0	0	0	0	
I-405	0	0	0	0	0	
US-30	0	0	0	0	0	
US-20	7	0	0	0	$1	
OR-38	2	1	1	0	$7	
OR-42	9	10	10	1	$118	
Others	61	5	1	0	$22	
Total	**123**	**33**	**23**	**2**	**$364**	**$94**

6.2 COMPARISON OF SCENARIOS

The figures and tables in the previous section show the estimated damage states of bridges and repair/replacement costs and delay-based user costs due to the traffic disruptions for the six scenarios considered.

A scenario earthquake at Portland Hills of a lower magnitude than all other scenarios showed the highest bridge repair/replacement cost. This is attributed to the fact that there is a majority of large bridges are concentrated in that area, hence repair/replacement costs is higher. Although the total number of damaged bridges is less than the subduction zone events, next to a magnitude 9 subduction scenario, the Portland Hills scenario caused a large number of collapses thereby resulting in more extensive damage.

The estimated number of damaged bridges and bridge repair/replacement cost for subduction zone earthquakes scenarios of magnitude 8.3 in the North and South is moderately similar. However, the cost of travel time loss varied greatly. These travel time related losses are inherently low and do not seem realistic. The unexpected values are likely attributed to the travel time loss estimation in REDARS, which has been primarily developed for smaller models and for urban areas. Nevertheless, the relative values are meaningful as the variation of the travel time loss of CSZ North M8.3 ($8 million) and CSZ South M8.3 ($94) can be attributed to the network redundancy in the northern part compared to the southern part of the state. The disruption of any of these links or nodes can cause a section of the network to go down, the impact of which is dependent on the redundancy in the system redundancy. The southern scenario caused a cluster of damages on US 101 and there is extensive damage to bridges lying on other routes where the traffic can be diverted such as OR 42 and OR 38. This reduces the redundancy of the system in the area. Similarly, even though similar number of bridges were damaged in the Northern scenario, the network is however more redundant.

The Klamath Falls and Scotts Mills M7.0 scenarios caused a relatively lower damage. This is because the epicenters of these earthquakes are in lightly populated areas compared to the Portland Hills scenario where there are a larger number of bridges in proximity to the epicenter.

7.0 SENSITIVITY ANALYSIS

Sensitivity analysis was conducted to determine how sensitive our model is to changes in the value of the parameters of the model such as soil profile and bridge fragility. These sensitivity investigations are performed as a series of analyses in which the soil profile and fragility values are changed to see how a change in the parameter causes a change in the overall outcome such as damage states and peak ground accelerations and spectral accelerations of the bridges. By showing how the model behavior responds to changes in the values, sensitivity analysis is a useful in model building and evaluation.

7.1 SOIL PROFILE

7.1.1 Liquefaction Settlement Trigger of Damage State

Liquefaction settlement trigger of damage state largely depends on the PGA. However, the sensitivity of the model for varying PGA only starts at a very low acceleration. For example, a bridge located at US101 (HWY009) over ELK RIVER has the following soil properties. Total overburden pressure = 642 kip/ft^2, effective overburden pressure = 174 kip/ft^2, thickness of layer is 15' and the blow count number is 1. For a PGA $\geq$ 0.00045g, the bridge analyses resulted in complete collapse. The PGA for larger PGA values then seems to only depend on the blow count number and thickness & difference between total and effective over burden pressure of the layer (Figure 12) p.22.

7.1.2 Earthquake Magnitude

The ground motion model by Abrahamson and Silva (1997) applies to shallow crustal earthquakes in active tectonic regions of the western United States. Abrahamson and Silva attenuation model expresses the ground motion as a function of the earthquake magnitude, source-site distance, local soil conditions, etc. The sensitivity soil profile on earthquake magnitude was analyzed for the five NEHRP site classifications A- E. For a magnitude 7 earthquake scenario at Scotts Mills area the PGAs and SAs were analyzed and compared for the different soil profiles. For soil profile A, B and C the results were identical. For soil profiles D and E, the results were again identical, but different from A, B and C. The following chart shows the analysis result for a bridge that is about 8 km away from the earthquake epicenter, if the site classification were A, B, or C the PGA, SA03 and SA10 would be 0.44g, 0.87g and 0.33g respectively and for site class D and E, the PGA, SA03 and SA10 were calculated to be 0.34g, 0.81g and 0.51g respectively (Figure 24).

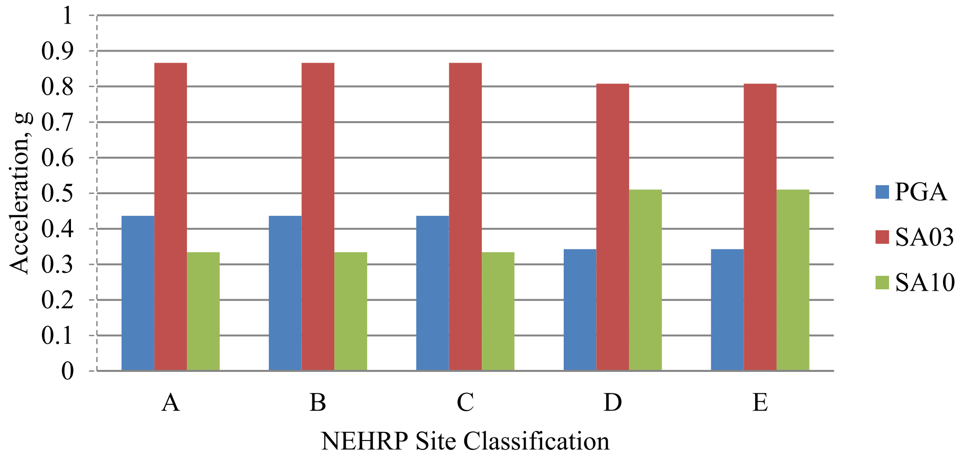

Figure 24: Site Classification Sensitivity on Earthquake magnitude

7.2 FRAGILITY

7.2.1 Fragility Curves

Bridge fragility curves are needed not only for seismic risk assessment, but also for uses in other activities such as bridge retrofit prioritization and post earthquake emergency response (Nielson 2005). REDARS2 provides default fragility curves originating from HAZUS (FEMA) model for all bridges in the system. These default values can however be replaced by a user-specified model for various individual bridges. Even though this would give a very refined result, it is impractical to implement to all the bridges in the study area. But it should be done for those bridges with unique configurations or whose seismic performance would have a significant effect on the ability of the roadway (Werner G-1). These fragility curves have been generated for typical bridge classes and not for individual bridges as reflected by the approach implemented in this study. The bridge classes are created and the individual bridges are grouped using the National Bridge Inventory (NBI) database as the basis. Medians of these damage functions are given in Table B. 1. These median values are modified to convert from standard to actual bridge. The factors accounted for are skew angle and three dimensional deck arching membrane.

In order to study the sensitivity of the model for the different fragilities, bridge fragilities on the major highway routes have been changed, one route at a time, and analysis results were compared. Figure 25 gives a comparison of fragility curves for a HWB4 (Seismically designed single span bridge) and HWB5 (Concrete, Multi-Column Bent, Simple Support) and as can be seen in the figure, a 50% probability of damage state 2 (Slight) for HWB5 is close to 0.25g where as HWB4 will not see damage until the spectral acceleration is 0.8g. Fragility curves for a long span bridge that crosses the Mississippi River along I-40 in Memphis, Tennessee were used as another extreme case in understanding the susceptibility of the routes if all the bridges in those routes were very prone to earthquake damage. These curves are based on the un-retrofitted configuration of the bridge that crosses Mississippi River along I-40 (Figure 26). The results achieved were then used in the assessment of the vulnerability of the highway routes and recommendation for retrofit. The number of bridges damaged for the different fragilities are shown below.

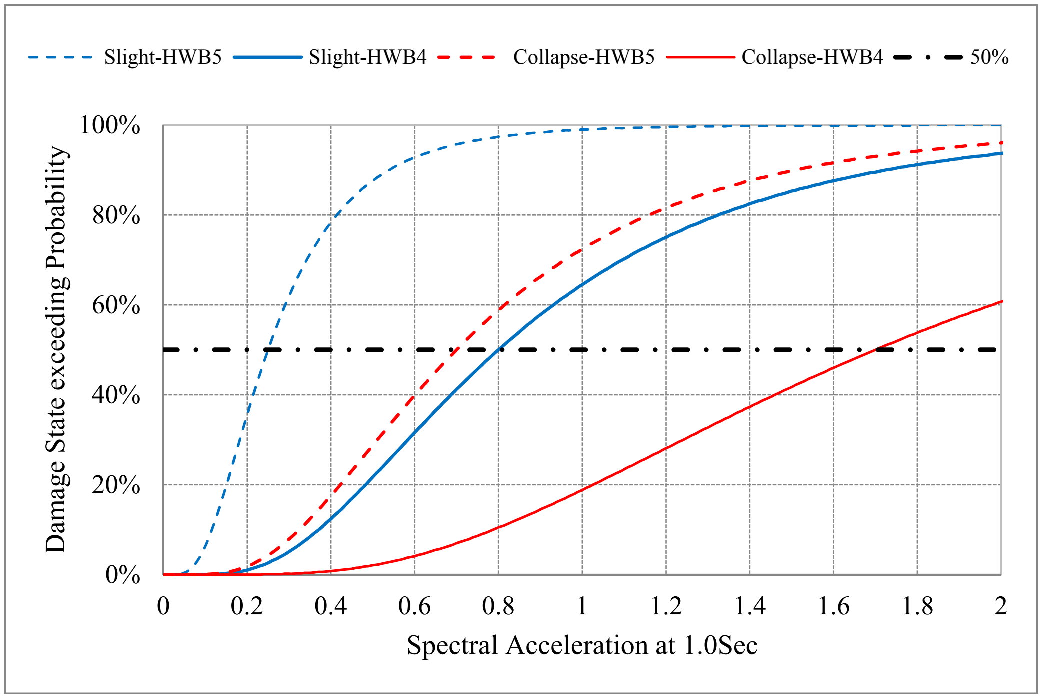

Figure 25: Fragility curve comparison of HWB4 and HWB5.

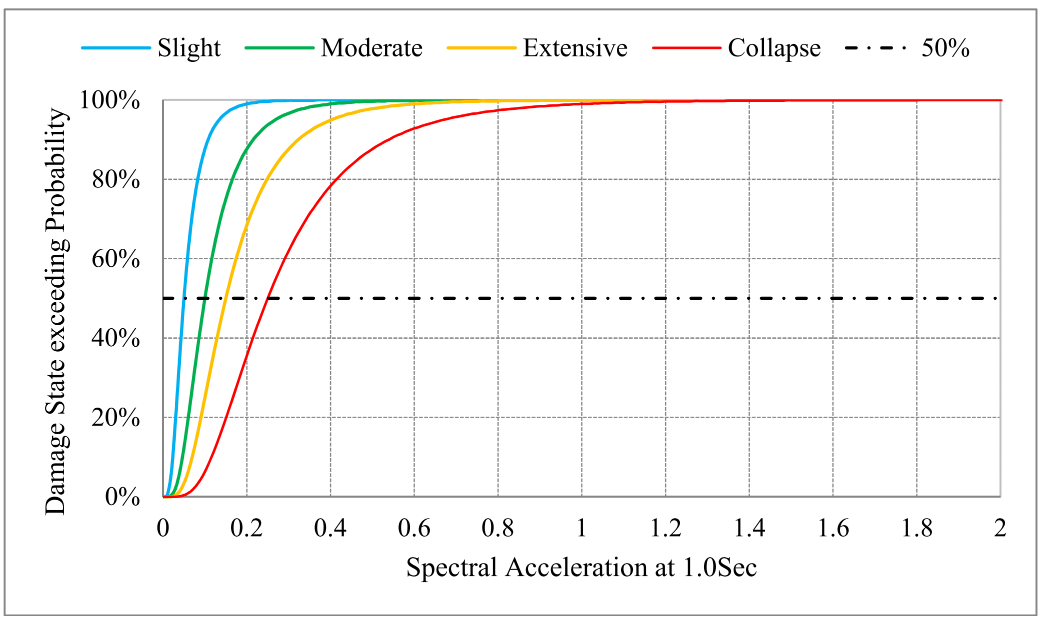

Figure 26: Aggregated fragility curve for I-40 crossing of Mississippi River.

Cost Analysis by Varying Bridge Fragility by Route

Table 16: Cascadia Subduction Zone (CSZ) scenario event M9.0

Route	Fragility	Damage states and Cost of entire network					
		slight	moderate	Extensive	Collapse	Repair Cost	Travel time cost
I-5 (MWC)	Existing	164	106	64	6	$1,080	$177
	HWB4	160	105	64	6	$1,073	$177
	I-40 MRC	160	190	76	6	$29,536	$177
I-5 (MLL)	Existing	164	106	64	6	$1,080	$177
	HWB4	148	103	6	6	$1,073	$177
	I-40 MRC	148	106	280	7	$2,620	$251
I-5 (DJJ)	Existing	164	106	64	6	$1,080	$177
	HWB4	137	106	64	6	$1,073	$177
	I-40 MRC	137	163	191	6	$2,235	$203
I-84	Existing	164	106	64	6	$1,080	$177
	HWB4	151	105	64	6	$1,070	$177
	I-40 MRC	205	166	65	6	$1,262	$177
US 101	Existing	164	106	64	6	$1,080	$177
	HWB4	157	92	28	1	$395	$48
	I-40 MRC	157	92	28	144	$2,397	$309
US 26	Existing	164	106	64	6	$1,080	$177
	HWB4	157	101	64	6	$1,071	$177
	I-40 MRC	163	124	95	22	$1,482	$3,855
I 205	Existing	164	106	64	6	$1,080	$177
	HWB4	156	104	64	6	$1,070	$177
	I-40 MRC	156	186	64	6	$1,626	$177
I 405	Existing	164	106	64	6	$1,080	$177
	HWB4	157	106	64	6	$1,079	$177
	I-40 MRC	157	162	64	6	$1,373	$177
US 30	Existing	164	106	64	6	$1,080	$177
	HWB4	159	104	62	6	$1,054	$177
	I-40 MRC	160	107	81	16	$1,470	$213
US 20	Existing	164	106	64	6	$1,080	$177
	HWB4	160	104	59	6	$1,062	$162
	I-40 MRC	160	104	59	28	$1,224	$201
OR 38	Existing	164	106	64	6	$1,080	$177
	HWB4	161	104	63	6	$1,071	$148
	I-40 MRC	161	104	72	15	$1,171	$180
OR 42	Existing	164	106	64	6	$1,080	$177
	HWB4	160	93	51	5	$922	$145
	I-40 MRC	160	93	60	50	$1,365	$243

Table 17: Cascadia Subduction zone scenario event- North M8.3

Route	Fragility	Damage states and Cost of entire network					
		slight	moderate	Extensive	Collapse	Repair Cost	Travel time cost
I-5 (MWC)	Existing	152	32	28	0	$337	$8
	HWB4	151	32	28	0	$337	$8
	I-40 MRC	151	129	28	0	$28,479	$8
I-5 (MLL)	Existing	152	32	28	0	$337	$8
	HWB4	134	31	28	0	$337	$8
	I-40 MRC	134	231	49	0	$818	$13
I-5 (DJJ)	Existing	152	32	28	0	$337	$8
	HWB4	149	32	28	0	$337	$8
	I-40 MRC	210	92	40	0	$590	$10
I-84	Existing	152	32	28	0	$337	$8
	HWB4	144	32	28	0	$334	$8
	I-40 MRC	190	80	28	0	$473	$8
US 101	Existing	152	32	28	0	$337	$8
	HWB4	145	14	9	0	$85	$3
	I-40 MRC	163	27	21	97	$1,769	$155
US 26	Existing	152	32	28	0	$337	$8
	HWB4	143	32	28	0	$335	$8
	I-40 MRC	150	66	49	12	$613	$3,554
I 205	Existing	152	32	28	0	$337	$8
	HWB4	148	32	28	0	$336	$8
	I-40 MRC	148	114	28	0	$890	$8
I 405	Existing	152	32	28	0	$337	$8
	HWB4	152	32	28	0	$337	$8
	I-40 MRC	152	88	28	0	$630	$8
US 30	Existing	152	32	28	0	$337	$8
	HWB4	148	30	26	0	$318	$8
	I-40 MRC	148	47	33	9	$606	$66
US 20	Existing	152	32	28	0	$337	$8
	HWB4	150	30	24	0	$323	$8
	I-40 MRC	150	30	24	22	$485	$77
OR 38	Existing	152	32	28	0	$337	$8
	HWB4	148	32	28	0	$336	$8
	I-40 MRC	148	39	30	0	$409	$10
OR 42	Existing	152	32	28	0	$337	$8
	HWB4	148	31	28	0	$332	$8
	I-40 MRC	148	85	28	0	$443	$8

Table 18: Cascadia Subduction zone scenario event- South M8.3

Route	Fragility	Damage states and Cost of entire network					
		slight	moderate	Extensive	Collapse	Repair Cost	Travel time cost
I-5 (MWC)	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	220	33	23	2	$3,741	$94
I-5 (MLL)	Existing	123	33	23	2	$364	$94
	HWB4	105	32	23	2	$359	$94
	I-40 MRC	113	245	23	2	$739	$94
I-5 (DJJ)	Existing	123	33	23	2	$364	$94
	HWB4	103	33	23	2	$360	$94
	I-40 MRC	104	175	64	2	$998	$104
I-84	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	167	33	23	2	$378	$94
US 101	Existing	123	33	23	2	$364	$94
	HWB4	116	17	12	1	$157	$33
	I-40 MRC	142	55	17	69	$1,247	$194
US 26	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	190	33	23	2	$381	$94
I 205	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	203	33	23	2	$430	$94
I 405	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	179	33	23	2	$399	$94
US 30	Existing	123	33	23	2	$364	$94
	HWB4	123	33	23	2	$364	$94
	I-40 MRC	138	33	23	2	$370	$94
US 20	Existing	123	33	23	2	$364	$94
	HWB4	116	17	12	1	$157	$33
	I-40 MRC	116	43	45	2	$461	$98
OR 38	Existing	123	33	23	2	$364	$94
	HWB4	121	32	22	2	$357	$76
	I-40 MRC	121	36	30	8	$437	$95
OR 42	Existing	123	33	23	2	$364	$94
	HWB4	114	23	13	1	$246	$73
	I-40 MRC	114	23	24	44	$688	$106

Table 19: Portland Hills scenario event- South M6.5

Route	Fragility	Damage states and Cost of entire network					
		slight	moderate	Extensive	Collapse	Repair Cost	Travel time cost
I-5 (MWC)	Existing	27	41	48	5	$1,573	$68
	HWB4	19	29	38	4	$1,090	$68
	I-40 MRC	19	40	54	74	$113,529	$96
I-5 (MLL)	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	46	49	48	5	$1,593	$68
I-5 (DJJ)	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	27	41	48	5	$1,573	$68
I-84	Existing	27	41	48	5	$1,573	$68
	HWB4	26	37	38	4	$1,090	$68
	I-40 MRC	37	53	49	23	$1,816	$70
US 101	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	33	41	48	5	$1,574	$68
US 26	Existing	27	41	48	5	$1,573	$68
	HWB4	24	38	41	5	$1,510	$61
	I-40 MRC	28	45	48	42	$1,981	$104
I 205	Existing	27	41	48	5	$1,573	$68
	HWB4	22	37	48	5	$1,560	$68
	I-40 MRC	22	55	112	5	$3,506	$77
I 405	Existing	27	41	48	5	$1,573	$68
	HWB4	24	29	39	3	$1,080	$64
	I-40 MRC	24	29	39	59	$2,255	$76
US 30	Existing	27	41	48	5	$1,573	$68
	HWB4	26	41	47	4	$1,452	$66
	I-40 MRC	31	43	53	11	$1,655	$149
US 20	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	27	41	48	5	$1,573	$68
OR 38	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	27	41	48	5	$1,573	$68
OR 42	Existing	27	41	48	5	$1,573	$68
	HWB4	27	41	48	5	$1,573	$68
	I-40 MRC	27	41	48	5	$1,573	$68

Travel time cost for I-5 northern segment (from Multnomah County to Clackamas County) is not affected when the fragility of the route was varied between two extremes cases. For a very weak fragility, there were a lot more slightly damaged bridges which would become operational after an event. This would be the reason why the travel time related losses are not significant. However, altering fragility of US-101 showed the highest variation in both repair –replacement cost and travel time related losses. There are a lot of extensive and complete collapse cases on this route and that would cause a major disruption in system performance. However, in the prioritization attempt, other factors, such as average daily traffic, length of segment were considered. The following chapter gives the algorithm employed. From the above results it can be seen that, highways routes that are least redundant are going to disrupt the system performance.

8.0 VULNERABILITY OF EXISTING NETWORK AND RECOMMENDATION FOR BRIDGE RETROFIT

8.1 VULNERABILITY OF OREGON HIGHWAY NETWORK

With a majority of state owned bridges designed and built between 1950 and 1980, the state of Oregon would face a devastating post earthquake situation if a major event occurred in the state. Even though retrofitting all vulnerable bridges in the near future might not be feasible, we can find ways to start moving in that direction. Therefore, the prioritization process of major highway segments, or key individual bridges, that are vulnerable under seismic loading will be important and necessary (*ODOT 2009*).

In order to estimate the economic loss of the major highway routes, all the earthquake scenarios considered were given similar chance of occurrence in the near future and the maximum cost incurred was considered for each highway segment. The economic losses include repair or replacement cost of bridges and the cost associated with travel time losses. These costs will be an important factor in determining the priority of each segment to be retrofitted. Retrofit cost is divided into two phases – Phase I and Phase II. Phase I retrofit includes replacing unstable bearings with stable bearings, providing additional seat width, limit movement of girders parallel to roadway using restrainers and limit movement of girders perpendicular to roadway using shear lugs. The main goal of Phase I retrofitting is "life safety." This is accomplished with retrofit details designed to prevent the superstructure from separating from the substructure and thereby preventing collapse of a span. Phase I retrofit is effective for moderate earthquakes. Phase II retrofit includes strengthening the substructure elements. The major goal of Phase II retrofitting is also "life safety" but for maximum anticipated earthquake. Equation 2 shows how these values are calculated. Using those cost estimates, the inventory replacement value of over 1900 bridges that are part of the Oregon State Highways system is about \$21 billion. Phase I retrofit cost is a little over \$1 billion and Phase II retrofit cost is about \$3 billion.

Equation 2:

Retrofit Cost (Phase I) = \$35/ft ^{2} x the deck area

Retrofit Cost (Phase II) = \$90/ft ^{2} x the deck area

Table 20 gives a breakdown of the distribution and replacement and retrofit cost of the bridges along the major highway routes.

Table 20: Replacement cost of state highway bridges on selected routes in millions.

Route	No of Bridges	Replacement Cost (in millions)	Retrofit cost(in millions)	
			Phase I	Phase II
I-5 (MWC)	97	$2,310	$130	$330
I-5 (MLL)	221	$765	$41	$105
I-5 (DJJ)	184	$840	$45	$115
I-84	200	$920	$50	$130
US-101	143	$1,430	$80	$200
US-26	76	$660	$35	$90
I-205	82	$2,150	$120	$300
I-405	56	$1,175	$65	$165
US-30	38	$160	$8	$20
US-20	32	$400	$20	$55
OR-38	18	$100	$5	$13
OR-42	54	$120	$6	$17

Approach for route retrofit selection strategy was based on "cost vs. benefit." The retrofit cost of considered routes is first estimated. Then the benefit of retrofitting each route is estimated by altering bridge fragility and re-running analysis. Of the major highway routes that were considered, US 101, OR 42, US 20 and OR 38 are the ones that showed an increase in travel time related losses. When the routes were replaced with stronger bridges the cost went down for repair/replacement and an even significant decrease in travel time loss. But because financial constraint is a major deciding aspect, retrofit cost and the cost implication of bridge replacement or repair are taken into account together with other factors such as traffic volume on route and length of route were also considered. Though improving longer stretches of highways with lower costs would be a key criterion in prioritizing the system, attention should be given to most populated areas of our state (e.g. I-5 North). ODOT, in their November 2009 report titled Seismic Vulnerability of Oregon State Highway Bridges, has considered both the Route Length and the Average Daily Traffic to be very important factors in their retrofit prioritization process. And similar approach is taken in this case as well.

Table 21 gives the results of the algorithm. I-405 is the route that is of highest priority. Even though I-405 is a short length highway, that route however has the third highest average daily traffic compared to the other routes. And most importantly, the cost of repairing damaged bridges if a major earthquake occurs more than three times the amount estimated to retrofit all bridges in the route. Same principle was followed when ranking the rest of the routes.

These results are only intended to reflect the vulnerability of the highway network and what precautions could be taken to be better prepared. Earthquakes, in addition to damaging the roadway system, can also damage buildings, contents, and lifeline infrastructure which are not considered to be part of this highway bridge vulnerability study.

8.2 ADVANTAGES OF RETROFITTING

Identification of structures in most need of retrofitting is not an easy task because we first need to be able to identify the hazard, analyze the vulnerability of selected priority structures to that hazard, and then fix those structures. And the cost of retrofitting must be balanced against risk which makes the decision even harder.

Benefit of retrofit can be computed as the value loss that can be avoided or reduced. The value loss criteria includes bridge damage, property loss, causalities and the traffic disruption that can be caused that can be avoided. Retrofitting can be very expensive and the decision to retrofit depends on political, social, and economic factors as well as engineering issues. Retrofit cost can be computed in present values as equal to the total future economic losses avoided from social cost and repair/restoration cost over the remaining bridge service lives. Estimated benefit is compared with the retrofit cost to investigate the benefit-cost ratio (*Zhou 2009*).

It is impossible to design or retrofit a structure to be "earthquake proof" to be totally safe near the epicenter of a large quake but structures should be designed or retrofitted for "life safety". Common sense and engineering judgment will be necessary in weighing the actual costs and benefits of retrofitting, against the risks of doing nothing. Also, the effect on the entire highway system must be kept in mind. The priority ranking given on Table 21 shall only be taken as a preliminary ranking. These routes need to be examined more and an advanced cost-benefit investigations should be conducted before decisions are made.

Table 21: Preliminary Route Seismic Retrofit Prioritization Ranking.

Route	CSZ M9.0	CSZ M8.3North	CSZ M8.3South	Portland Hills M6.5	Retrofit cost		Total Cost (Repair & travel time)	Avg. Daily traffic	Route Length	Priority List
	Bridge Damage Cost (in millions)	Bridge Damage Cost (in millions)	Bridge Damage Cost (in millions)	Bridge Damage Cost (in millions)	Phase I (In Millions)	Phase II (In Millions)				
I-405	$2	$0	$0	$494	$65	$166	$562	157,560	4.21	**1**
I-5 (MWC)	$8	$0.4	$0	$483	$128	$328	$551	153,000	27.84	**2**
I-84	$10	$3	$0	$143	$50	$129	$211	177,000	149.7	**3**
US -101	$685	$252	$207	$0	$77	$197	$862	29,900	363.1	**4**
I-205	$10	$1	$0	$14	$118	$304	$187	168,100	27.18	**5**
OR-42	$158	$5	$118	$0	$6	$17	$335	23,300	77.17	**6**
I-5 (MLL)	$14	$5	$5	$0	$41	$105	$191	86,700	77.26	**7**
US-26	$9	$2	$0	$63	$35	$90	$186	138,020	128.9	**8**
US-20	$19	$14	$1	$0	$22	$55	$196	35,400	76.8	**9**
US-30	$26	$18	$0	$122	$8	$20	$203	29,406	99.34	**10**
I-5 (DJJ)	$5	$0.3	$4	$0	$45	$116	$182	48,200	203.6	**11**
OR-38	$9	$1	$7	$0	$5	$13	$186	6,400	57.23	**12**
Total Travel Time Cost (in millions)	$177	$8	$94	$68						

9.0 CONCLUDING REMARKS

9.1 SUMMARY

9.1.1 Earthquake Impacts on Highways

Bridges represent vulnerability points within a transportation network. Hence, damaged bridges will have a great impact on the system performance causing severe traffic congestion statewide. This disruption of traffic flow will in turn impact post-earthquake emergency response, repair and reconstruction operations.

Earthquake damage to multiple bridges would disable entire routes for up to three to six months and severe traffic congestion will occur for at least a year. Movement of goods to final destinations – for example, manufacturers, retail outlets, and hospitals – will be much slower for a long period of time. This will have long term economic consequences due to the valued loss of time when commuter and freight travel slows down due to the disrupted network. A commute to work that took 30 minutes could take hours; and businesses will suffer due to this disruption and may even move away from Oregon elsewhere.

In the scenarios considered, highways I-5, I-84, US-101, I-405, US-26, US-20, US-30, OR-38, and OR-42 have extensive damage or collapse. Other bridges, such as those owned by county or city are not currently incorporated into the model are also likely to experience damage. Failure of the roadways access due to other earthquake related effects including potential landslides and liquefaction were also not considered part of the scenario analyses.

9.1.2 Recovery Issues

Single bridges on some major routes may be replaced with in a year. However, it will probably take over 5 years to replace 70+ bridges due to limited resources. Another challenge is redirecting up to 150,000 cars per day (I-5) onto surface streets and that some streets cannot carry the increased traffic volumes that could possibly be diverted to them. And simultaneous route outages could bring traffic to a standstill, with few substitutes to carry daily traffic.

9.1.3 Analysis results and interpretation

Damage states of bridges are computed by first computing the bridge's demand spectral acceleration for a given scenario earthquake, it is then compared to each bridge's spectral acceleration capacity that leads to the onset of each damage state. However, these median values of ground motion computed do not necessarily represent the exact levels of ground shaking at the bridge locations since the exact levels of ground shaking of an earthquake will not be known without actually recording the motion with strong motion accelerators at the time of the event. Consequently, there is a probability that some bridges might perform better or worse during a real earthquake compared to a scenario analysis.

Previously developed analytical fragility curves are based on simplified models and simplified methodologies, which by their very nature include a significant amount of uncertainty, and therefore do not completely represent the performance of most bridges. To adequately represent the fragility of a bridge and to improve the reliability and effectiveness of seismic Risk assessment tools, improved fragility curves for highway bridges are needed.

In addition, fragility values are based on probabilistic median expected performances. A particular bridge that had a specific damage state may not exactly correlate to actual events but is more representative as the expected damage state. For these reasons, the aggregate response over the route should be examined and is more informative than considering the damage state of an individual bridge.

Furthermore, other devastating effects of earthquakes, such as potential landslides and liquefaction, are not included in these results. These analyses should be done outside or REDARS2 since REDARS2 doesn't have the capability of analyzing damaged caused due to landslides and the results from liquefaction are not well represented.

9.2 SUMMARY OF ISSUES ENCOUNTERED IN REDARS2

Implementing REDARS2 presents challenges. Following are some of the problems that were encountered

- Data needed were not easily accessible and often needed to be re-formatted.
- Even after large soil data collection, liquefaction assessment was not well represented.
- Use of "Import Wizard" is challenging and limiting for incorporating state wide size models and networks created from sources other than NHPMS.
- Bridge data needed to be manually defined as the import mechanism often left out bridges from the highways segment definitions
- Errors in bridge replacement cost estimation are inherent within the analyses and need to be post-processed.
- Inconsistent skew angle definitions and implementation was found in the NBI.

In all, rigorous input data validation process is needed when constructing REDARS2 models before analyses can be initiated. The input data validation was conducted in this study based on data obtained from Oregon DOT.

9.3 SUGGESTION FOR FUTURE RESEARCH

The work in the present study should be extended through additional research in the following areas:

- Develop improved fragility curves adequately represent the fragility of a bridge and to improve the reliability and effectiveness of seismic risk assessment tools, improved fragility curves for Oregon highway bridges are needed.

- Methodologies for the incorporation of liquefaction and ground deformation hazards should be developed to combine with the results of this study for a more comprehensive prioritization assessment.

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11.0 APPENDICES

11.1 APPENDIX A - HIGHWAY SYSTEM CLASSIFICATION

Table A. 1: Highway System Classification (*HAZUS-MH MR4 Technical Manual 2009*)

Label	Description
HWB1	Major Bridge - Length > 150m (Conventional Design)
HWB2	Major Bridge - Length > 150m (Seismic Design)
HWB3	Single Span – (Not HWB1 or HWB2) (Conventional Design)
HWB4	Single Span – (Not HWB1 or HWB2) (Seismic Design)
HWB5	Concrete, Multi-Column Bent, Simple Support (Conventional Design), Non-California (Non-CA)
HWB6	Concrete, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB7	Concrete, Multi-Column Bent, Simple Support (Seismic Design)
HWB8	Continuous Concrete, Single Column, Box Girder (Conventional Design)
HWB9	Continuous Concrete, Single Column, Box Girder (Seismic Design)
HWB10	Continuous Concrete, (Not HWB8 or HWB9) (Conventional Design)
HWB11	Continuous Concrete, (Not HWB8 or HWB9) (Seismic Design)
HWB12	Steel, Multi-Column Bent, Simple Support (Conventional Design), Non-California (Non-CA)
HWB13	Steel, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB14	Steel, Multi-Column Bent, Simple Support (Seismic Design)
HWB15	Continuous Steel (Conventional Design)
HWB16	Continuous Steel (Seismic Design)
HWB17	PS Concrete Multi-Column Bent, Simple Support - (Conventional Design), Non-California
HWB18	PS Concrete, Multi-Column Bent, Simple Support (Conventional Design), California (CA)
HWB19	PS Concrete, Multi-Column Bent, Simple Support (Seismic Design)
HWB20	PS Concrete, Single Column, Box Girder (Conventional Design)
HWB21	PS Concrete, Single Column, Box Girder (Seismic Design)
HWB22	Continuous Concrete, (Not HWB20/HWB21) (Conventional Design)
HWB23	Continuous Concrete, (Not HWB20/HWB21) (Seismic Design)
HWB24	Same definition as HWB12 except that the bridge length is less than 20 meters
HWB25	Same definition as HWB13 except that the bridge length is less than 20 meters
HWB26	Same definition as HWB15 except that the bridge length is less than 20 meters and Non-CA
HWB27	Same definition as HWB15 except that the bridge length is less than 20 meters and in CA
HWB28	All other bridges that are not classified (including wooden bridges)

11.2 APPENDIX B - DAMAGE ALGORITHMS AND FRAGILITY CURVES FOR BRIDGES

Table B. 1: Damage Algorithms for Bridges (*HAZUS-MH MR4 Technical Manual 2009*)

	Sa [1.0 sec in g's] for Damage Functions due to Ground Shaking				PGD [inches] for Damage Functions due to Ground Failure			
CLASS	Slight	Moderate	Extensive	Complete	Slight	Moderate	Extensive	Complete
HWB1	0.4	0.5	0.7	0.9	3.9	3.9	3.9	13.8
HWB2	0.6	0.9	1.1	1.7	3.9	3.9	3.9	13.8
HWB3	0.8	1	1.2	1.7	3.9	3.9	3.9	13.8
HWB4	0.8	1	1.2	1.7	3.9	3.9	3.9	13.8
HWB5	0.25	0.35	0.45	0.7	3.9	3.9	3.9	13.8
HWB6	0.3	0.5	0.6	0.9	3.9	3.9	3.9	13.8
HWB7	0.5	0.8	1.1	1.7	3.9	3.9	3.9	13.8
HWB8	0.35	0.45	0.55	0.8	3.9	3.9	3.9	13.8
HWB9	0.6	0.9	1.3	1.6	3.9	3.9	3.9	13.8
HWB10	0.6	0.9	1.1	1.5	3.9	3.9	3.9	13.8
HWB11	0.9	0.9	1.1	1.5	3.9	3.9	3.9	13.8
HWB12	0.25	0.35	0.45	0.7	3.9	3.9	3.9	13.8
HWB13	0.3	0.5	0.6	0.9	3.9	3.9	3.9	13.8
HWB14	0.5	0.8	1.1	1.7	3.9	3.9	3.9	13.8
HWB15	0.75	0.75	0.75	1.1	3.9	3.9	3.9	13.8
HWB16	0.9	0.9	1.1	1.5	3.9	3.9	3.9	13.8
HWB17	0.25	0.35	0.45	0.7	3.9	3.9	3.9	13.8
HWB18	0.3	0.5	0.6	0.9	3.9	3.9	3.9	13.8
HWB19	0.5	0.8	1.1	1.7	3.9	3.9	3.9	13.8
HWB20	0.35	0.45	0.55	0.8	3.9	3.9	3.9	13.8
HWB21	0.6	0.9	1.3	1.6	3.9	3.9	3.9	13.8
HWB22	0.6	0.9	1.1	1.5	3.9	3.9	3.9	13.8
HWB23	0.9	0.9	1.1	1.5	3.9	3.9	3.9	13.8
HWB24	0.25	0.35	0.45	0.7	3.9	3.9	3.9	13.8
HWB25	0.3	0.5	0.6	0.9	3.9	3.9	3.9	13.8
HWB26	0.75	0.75	0.75	1.1	3.9	3.9	3.9	13.8
HWB27	0.75	0.75	0.75	1.1	3.9	3.9	3.9	13.8
HWB28	0.8	1	1.2	1.7	3.9	3.9	3.9	13.8

Table B. 2: Median Ground Motions Leading to Onset of Various Damage States for Conventionally Designed "Standard" Bridges. (*REDARS2 Technical Manual, 2006*)

Bridge Type	NBI Class	Damage State	Median Spectral Acceleration, g, at Period = 1.0 sec. for Damage Functions due to Ground Shaking	
			Non- California	California
Single Span	All	2	0.80*[1]	0.80*
		3	1.00	1.00
		4	1.20	1.20
		5	1.70	1.70
Major Bridges	All	2	0.40	0.40
		3	0.50	0.50
		4	0.70	0.60
		5	0.90	0.90
Multi-Column Bents and Simply-Supported Concrete Superstructure	101-106 501-506	2	0.25	0.30
		3	0.35	0.50
		4	0.45	0.60
		5	0.70	0.90
Single-Column Bents and Concrete Box-Girder Superstructure	205-206 605-606	2	Not Applicable	0.35
		3		0.45
		4		0.55
		5		0.80
Continuous Reinforced-Concrete Superstructure	201-204	2	0.60*	0.90*
		3	0.90	0.90
		4	1.10	1.10
		5	1.50	1.50
Continuous Prestressed-Concrete Superstructure	601-604 607	2	0.60*	0.90*
		3	0.90	0.90
		4	1.10	1.10
		5	1.50	1.50
Simply-Supported Steel Superstructure	301-310	2	0.25	0.30
		3	0.35	0.50
		4	0.45	0.60
		5	0.70	0.90
Continuous Steel Superstructure	402-410	2	0.75*	0.75*
		3	0.75	0.75
		4	0.75	0.75
		5	1.10	1.10
Continuous Steel Superstructure	All	2	0.80	0.80
		3	1.00	1.00
		4	1.20	1.20
		5	1.70	1.70

[1] *Short period motions govern; therefore use demand and capacity at 0.3 sec. to assess damage state.

Table B. 3: Median Ground Motions Leading to Onset of Various Damage States for Seismically Designed "Standard" Bridges. (*REDARS2 Technical Manual, 2006*)

Bridge Type	NBI Class	Damage State	Median Spectral Acceleration, g, at Period = 1.0 sec. for Damage	
			Non- California	California
Single Span	All	2	0.80*[2]	0.80*
		3	1.00	1.00
		4	1.20	1.20
		5	1.70	1.70
Major Bridges	All	2	0.60	0.60
		3	0.90	0.90
		4	1.10	1.10
		5	1.70	1.70
Multi-Column Bents and Simply-Supported Concrete Superstructure	101-106 501-506	2	0.50	0.50
		3	0.80	0.80
		4	1.10	1.10
		5	1.07	1.70
Single-Column Bents and Concrete Box-Girder Superstructure	205-206 605-606	2	Not Applicable	0.60
		3		0.90
		4		1.30
		5		1.60
Continuous Reinforced-Concrete Superstructure	201-204	2	0.90*	0.90*
		3	0.90	0.90
		4	1.10	1.10
		5	1.50	1.50
Continuous Prestressed-Concrete Superstructure	601-604 607	2	0.90*	0.90*
		3	0.90	0.90
		4	1.10	1.10
		5	1.50	1.50
Simply-Supported Steel Superstructure	301-310	2	0.50	0.50
		3	0.80	0.80
		4	1.10	1.10
		5	1.07	1.07
Continuous Steel Superstructure	402-410	2	0.90*	0.90*
		3	0.90	0.90
		4	1.10	1.10
		5	1.50	1.50
Continuous Steel Superstructure	All	2	0.80	0.80
		3	1.00	1.00
		4	1.20	1.20
		5	1.70	1.70

[2] *Short period motions govern; therefore use demand and capacity at 0.3 sec. to assess damage state.

Table B. 4: HAZUS Bridge Classification Scheme

Class	NBI Class	State	Year built	# of Span	Length of max Span (meters)	Length less than 20m	Design	Description
HWB1	**All**	**Non-CA**	**< 1990**		**>150**	**N/A**	**Conventional**	**Major Bridge - Length > 150m**
HWB1	**All**	**CA**	**< 1975**		**>150**	**N/A**	**Conventional**	**Major Bridge - Length > 150m**
HWB2	**All**	**Non-CA**	**≥ 1990**		**>150**	**N/A**	**Seismic**	**Major Bridge - Length > 150m**
HWB2	**All**	**CA**	**≥ 1975**		**>150**	**N/A**	**Seismic**	**Major Bridge - Length > 150m**
HWB3	**All**	**Non-CA**	**< 1990**	**1**		**N/A**	**Conventional**	**Single Span**
HWB3	**All**	**CA**	**< 1975**	**1**		**N/A**	**Conventional**	**Single Span**
HWB4	**All**	**Non-CA**	**≥ 1990**	**1**		**N/A**	**Seismic**	**Single Span**
HWB4	**All**	**CA**	**≥ 1975**	**1**		**N/A**	**Seismic**	**Single Span**
HWB5	**101-106**	**Non-CA**	**< 1990**			**N/A**	**Conventional**	**Multi-Col. Bent Simple Support -concrete**
HWB6	**101-106**	**CA**	**< 1975**			**N/A**	**Conventional**	**Multi-Col. Bent Simple Support -concrete**
HWB7	**101-106**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Multi-Col. Bent Simple Support -concrete**
HWB7	**101-106**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Multi-Col. Bent Simple Support -concrete**
HWB8	**205-206**	**CA**	**< 1975**			**N/A**	**Conventional**	**Single Col. Box Girder - C Concrete**
HWB9	**205-206**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Single Col. Box Girder - C Concrete**
HWB10	**201-206**	**Non-CA**	**< 1990**			**N/A**	**Conventional**	**Continuous Concrete**

Table B. 5: HAZUS Bridge Classification Scheme (Continued)

Class	NBI Class	State	Year built	# of Span	Length of max Span (meters)	Length less than 20m	Design	Description
HWB10	**201-206**	**CA**	**< 1975**			**N/A**	**Conventional**	**Continuous Concrete**
HWB11	**201-206**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Continuous Concrete**
HWB11	**201-206**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Continuous Concrete**
HWB12	**301-306**	**Non-CA**	**< 1990**			**No**	**Conventional**	**Multi-Col. Bent Simple Support -Steel**
HWB13	**301-306**	**CA**	**< 1975**			**No**	**Conventional**	**Multi-Col. Bent Simple Support -Steel**
HWB14	**301-306**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Multi-Col. Bent Simple Support -Steel**
HWB14	**301-306**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Multi-Col. Bent Simple Support -Steel**
HWB15	**402-410**	**Non-CA**	**< 1990**			**No**	**Conventional**	**Continuous Steel**
HWB15	**402-410**	**CA**	**< 1975**			**No**	**Conventional**	**Continuous Steel**
HWB16	**402-410**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Continuous Steel**
HWB16	**402-410**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Continuous Steel**
HWB17	**501-506**	**Non-CA**	**< 1990**			**N/A**	**Conventional**	**Multi-Col. Bent, Simple Support – Prestressed Concrete**
HWB18	**501-506**	**CA**	**< 1975**			**N/A**	**Conventional**	**Multi-Col. Bent, Simple Support – Prestressed Concrete**
HWB19	**501-506**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Multi-Col. Bent, Simple Support – Prestressed Concrete**
HWB19	**501-506**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Multi-Col. Bent, Simple Support – Prestressed Concrete**

Table B. 6: HAZUS Bridge Classification Scheme (Continued)

Class	NBI Class	State	Year built	# of Span	Length of max Span (meters)	Length less than 20m	Design	Description
HWB20	**605-606**	**CA**	**< 1975**			**N/A**	**Conventional**	**Single-Col. Bent, Simple Support – Prestressed Concrete**
HWB21	**605-606**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Single-Col. Bent, Simple Support – Prestressed Concrete**
HWB22	**601-607**	**Non-CA**	**< 1990**			**N/A**	**Conventional**	**Continuous Concrete**
HWB22	**601-607**	**CA**	**< 1975**			**N/A**	**Conventional**	**Continuous Concrete**
HWB23	**601-607**	**Non-CA**	**≥ 1990**			**N/A**	**Seismic**	**Continuous Concrete**
HWB23	**601-607**	**CA**	**≥ 1975**			**N/A**	**Seismic**	**Continuous Concrete**
HWB24	**301-306**	**Non-CA**	**< 1990**			**Yes**	**Conventional**	**Multi-Col. Bent Simple support - Steel**
HWB25	**301-306**	**CA**	**< 1975**			**Yes**	**Conventional**	**Multi-Col. Bent Simple support - Steel**
HWB26	**402-410**	**Non-CA**	**< 1990**			**Yes**	**Conventional**	**Continuous Steel**
HWB27	**402-410**	**CA**	**< 1975**			**Yes**	**Conventional**	**Continuous Steel**
HWB28	**All**							**Other bridges that are not classified**

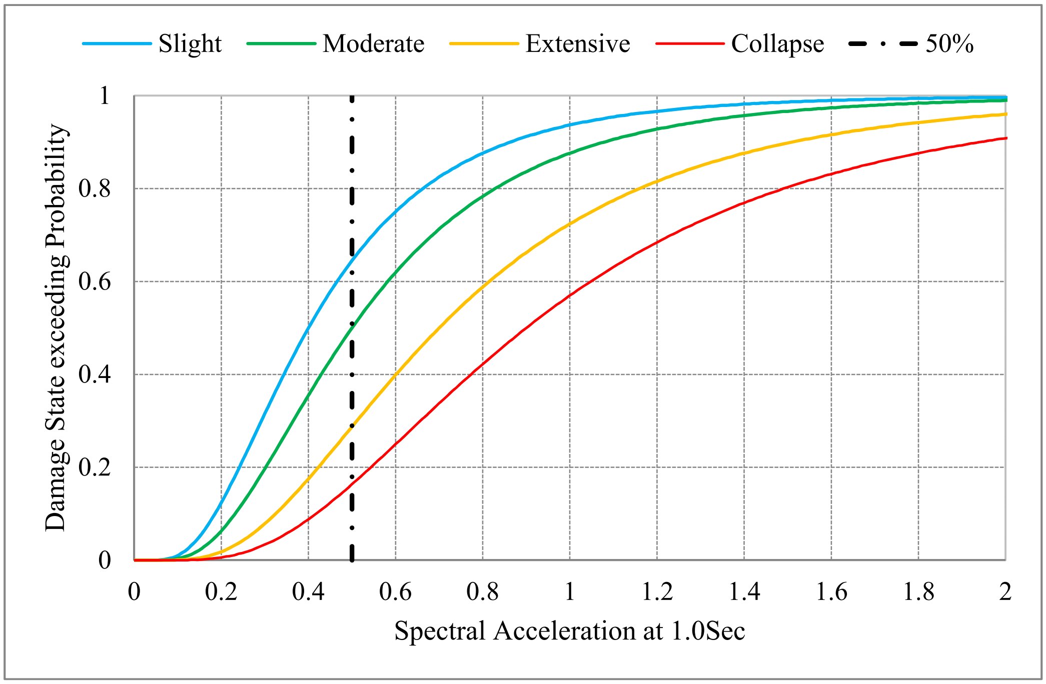

Figure B. 1: Fragility Curves for Conventionally Designed Major Bridges (HWB1).

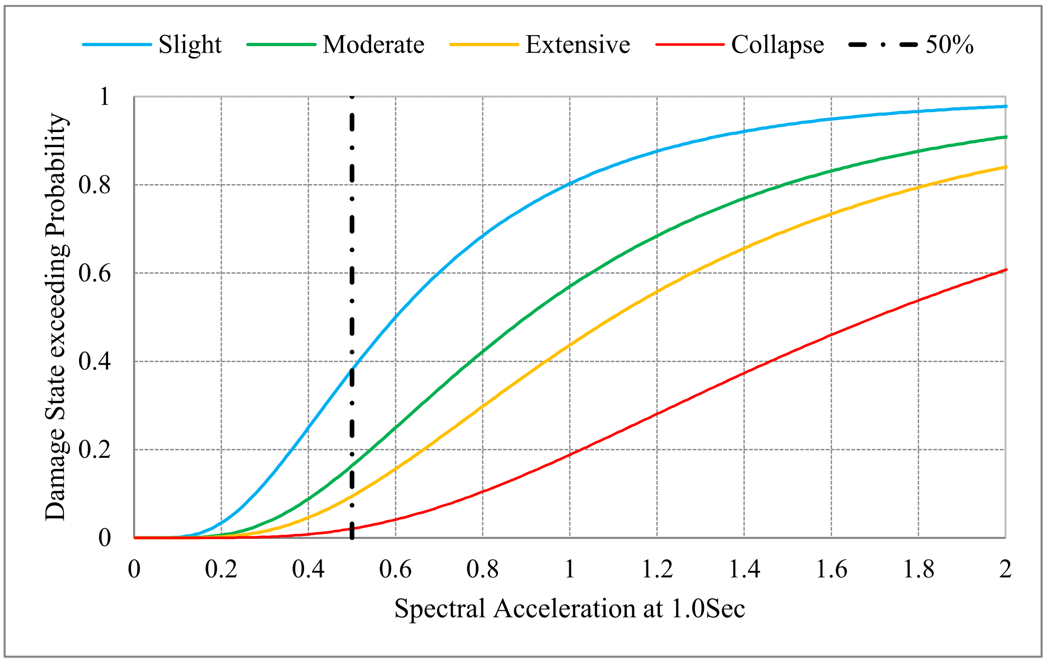

Figure B. 2: Fragility Curves for Conventionally Designed Major Bridges (HWB2).

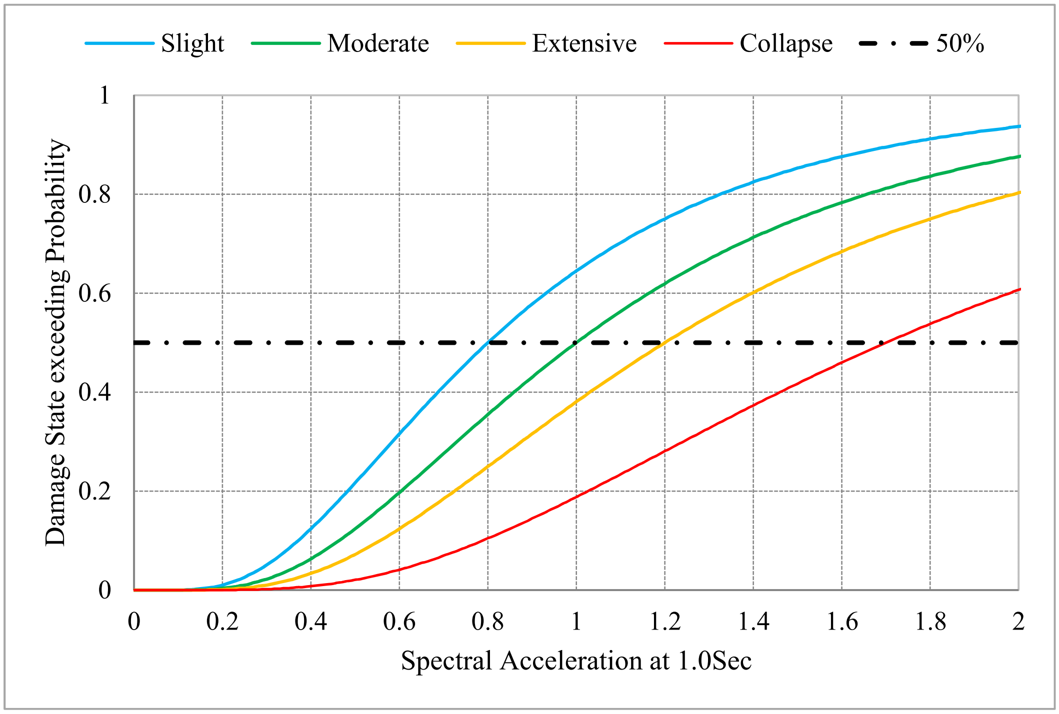

Figure B. 3: Fragility Curves for Conventionally Designed Major Bridges (HWB3, 4 & 18).

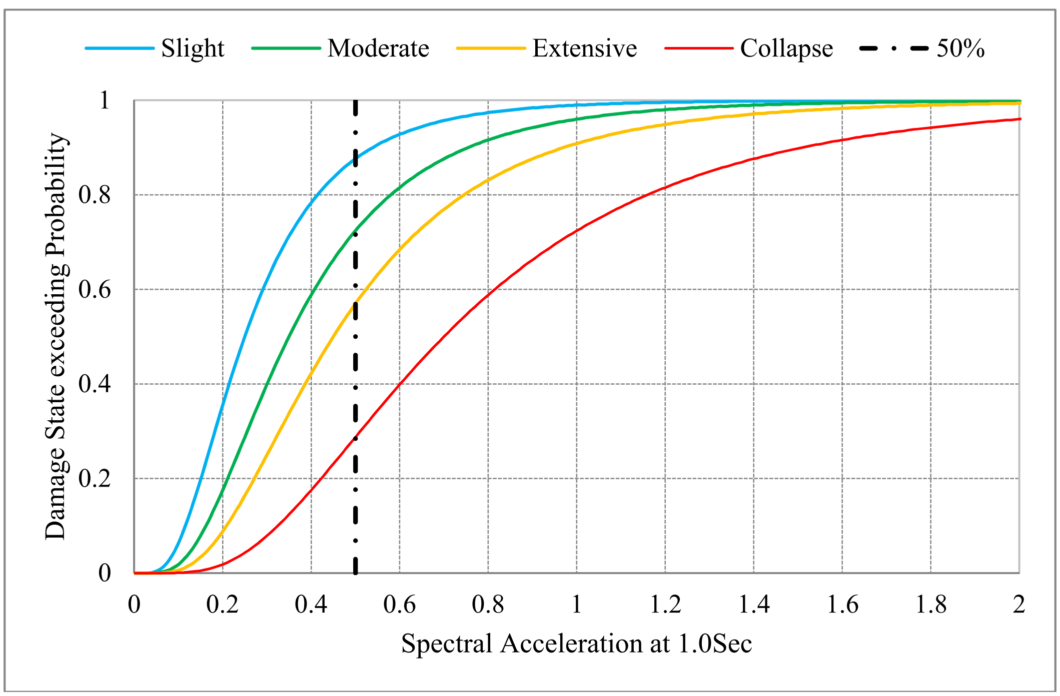

Figure B. 4: Fragility Curves for Conventionally Designed Major Bridges (HWB5, 12, 17 & 24).

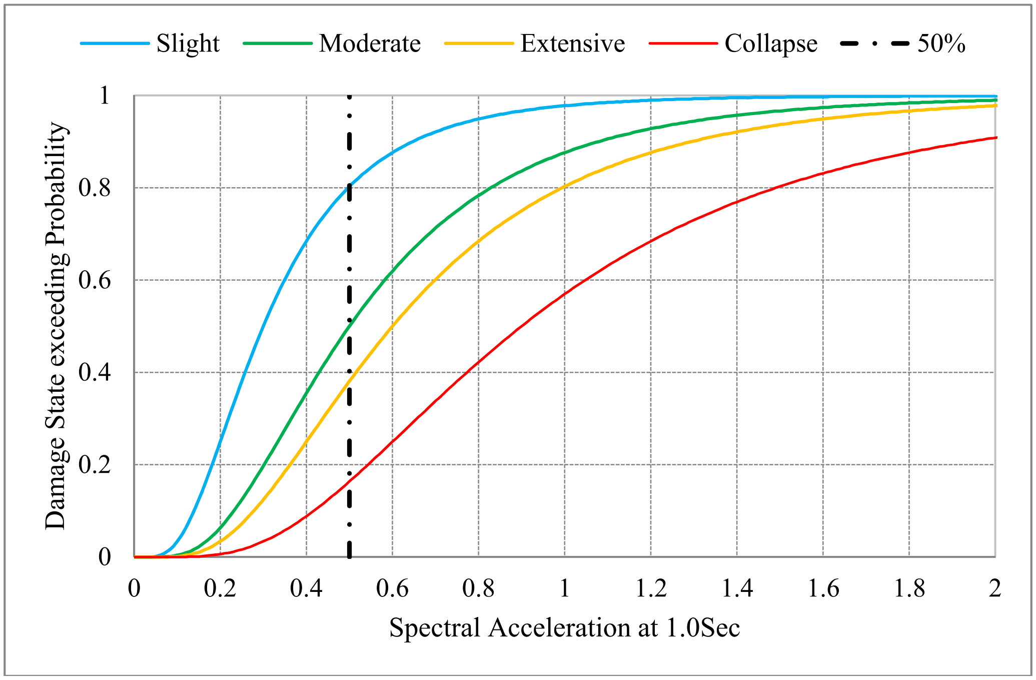

Figure B. 5: Fragility Curves for Conventionally Designed Major Bridges (HWB6, 13, 18 & 25).

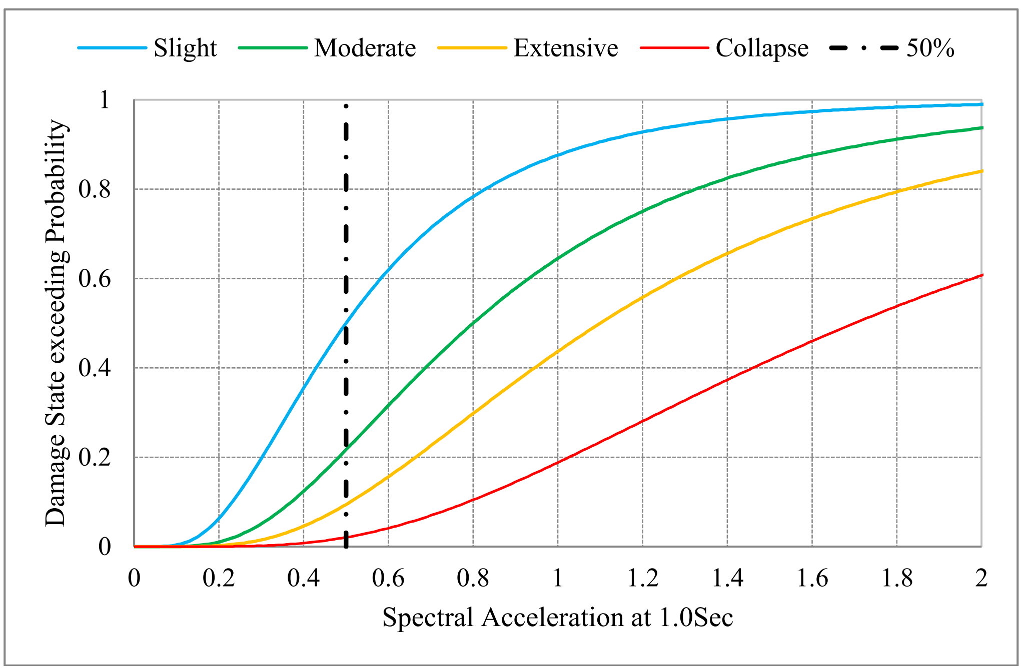

Figure B. 6: Fragility Curves for Conventionally Designed Major Bridges (HWB7, 14 & 19).

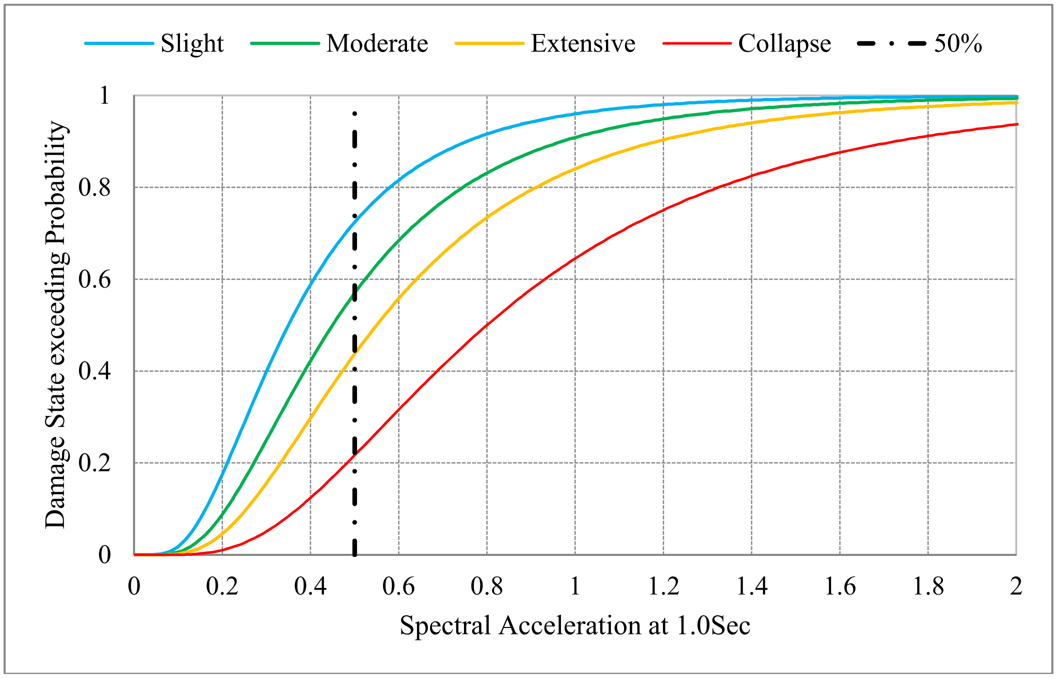

Figure B. 7: Fragility Curves for Conventionally Designed Major Bridges (HWB8 & HWB20).

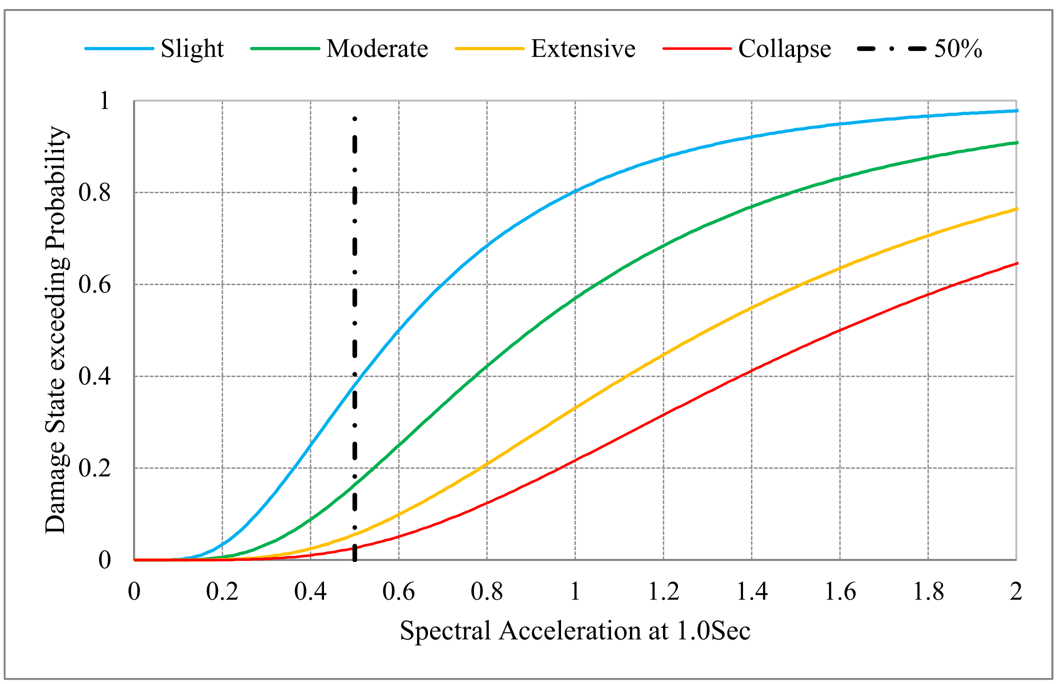

Figure B. 8: Fragility Curves for Conventionally Designed Major Bridges (HWB9 & HWB21).

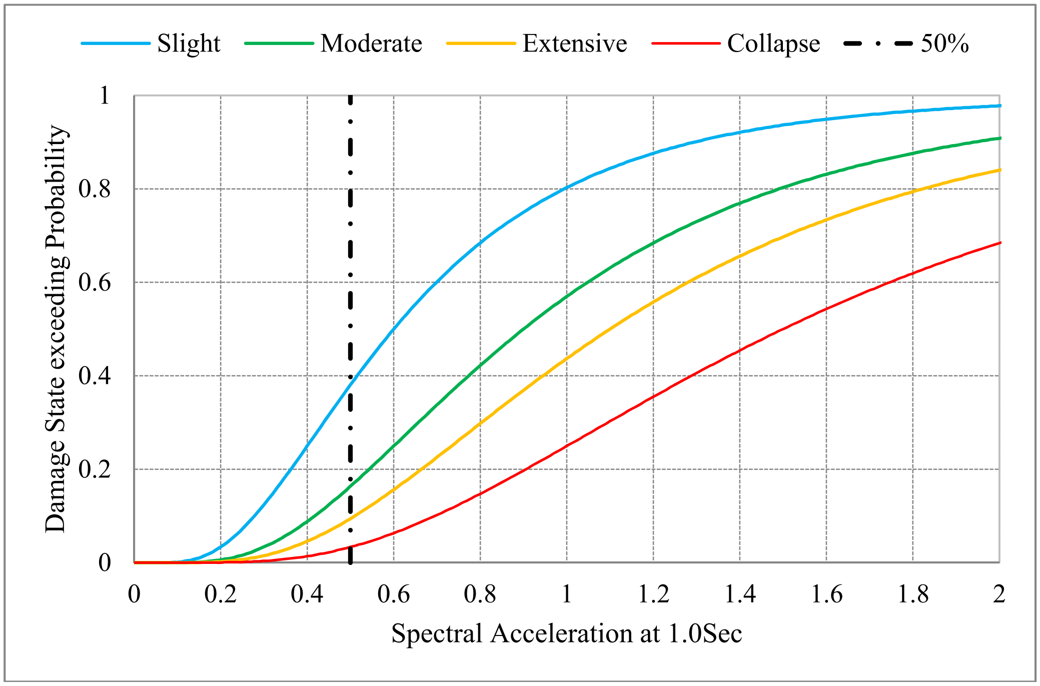

Figure B. 9: Fragility Curves for Conventionally Designed Major Bridges (HWB10, 11, 22 & 23).

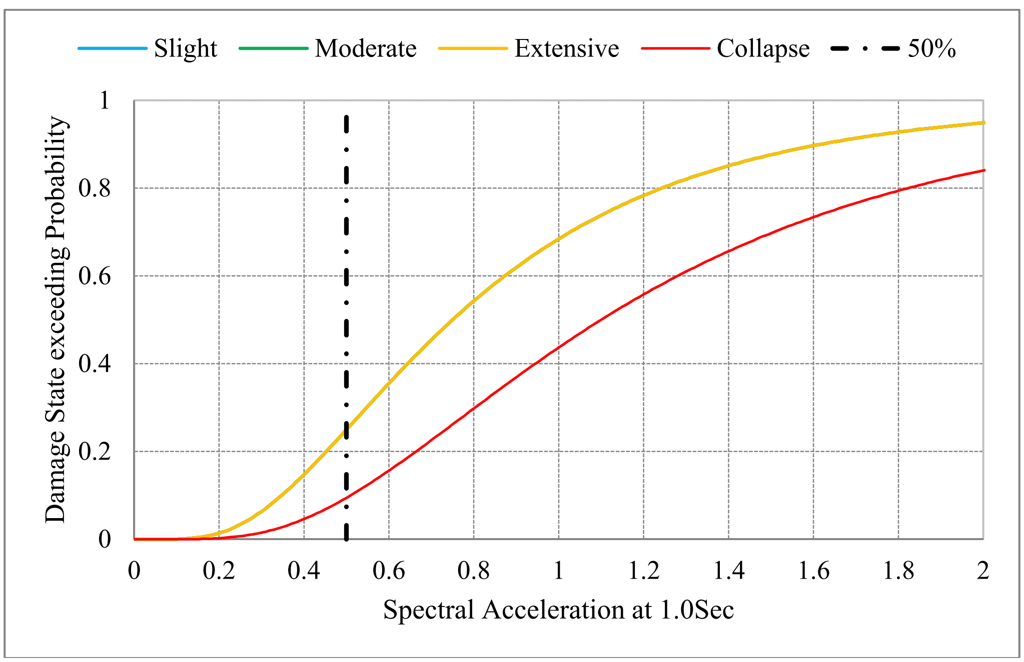

Figure B. 10: Fragility Curves for Conventionally Designed Major Bridges (HWB15, 26 & 27).

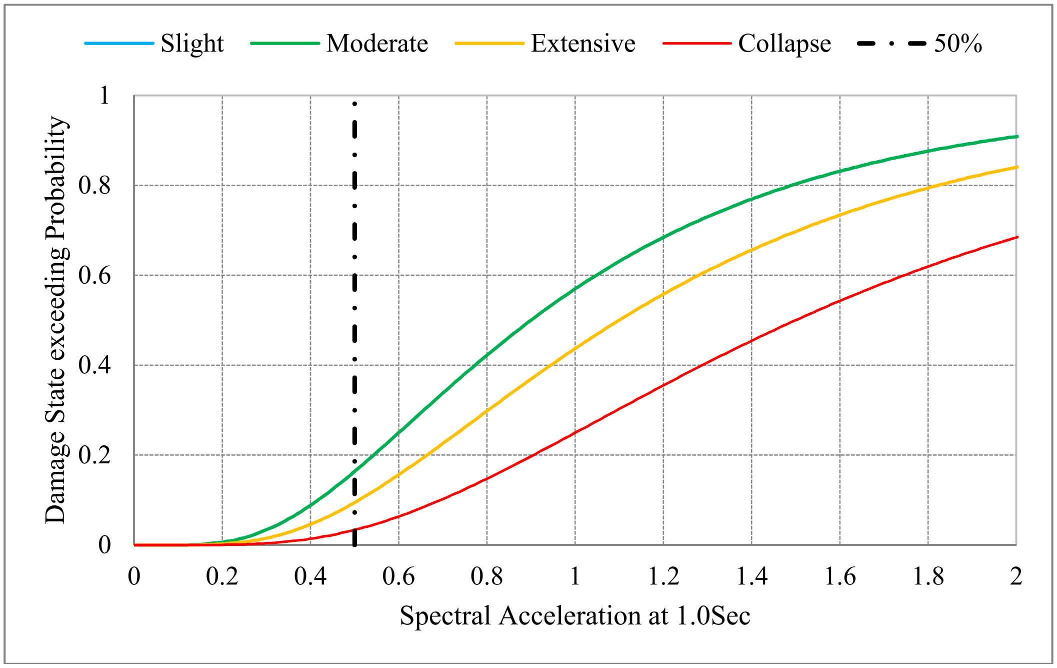

Figure B. 11: Fragility Curves for Conventionally Designed Major Bridges (HWB16).

11.3 APPENDIX C - SCENARIO RESULTS BY HIGHWAY ROUTE AND BRIDGE TYPE

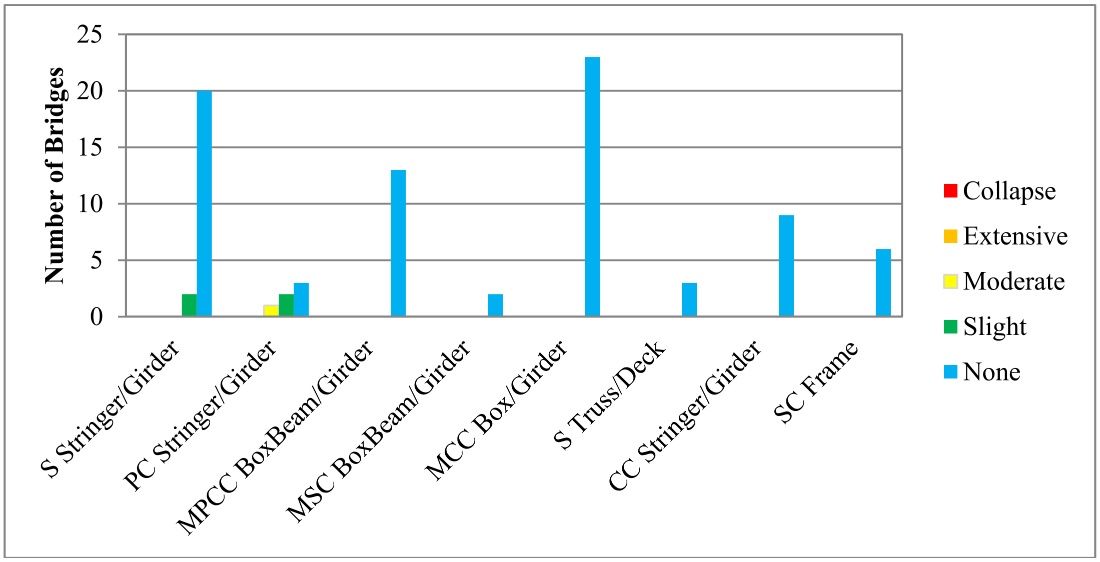

Figure C. 1: Cascadia M9.0 Scenario on I-5 from Multnomah to Clackamas County.

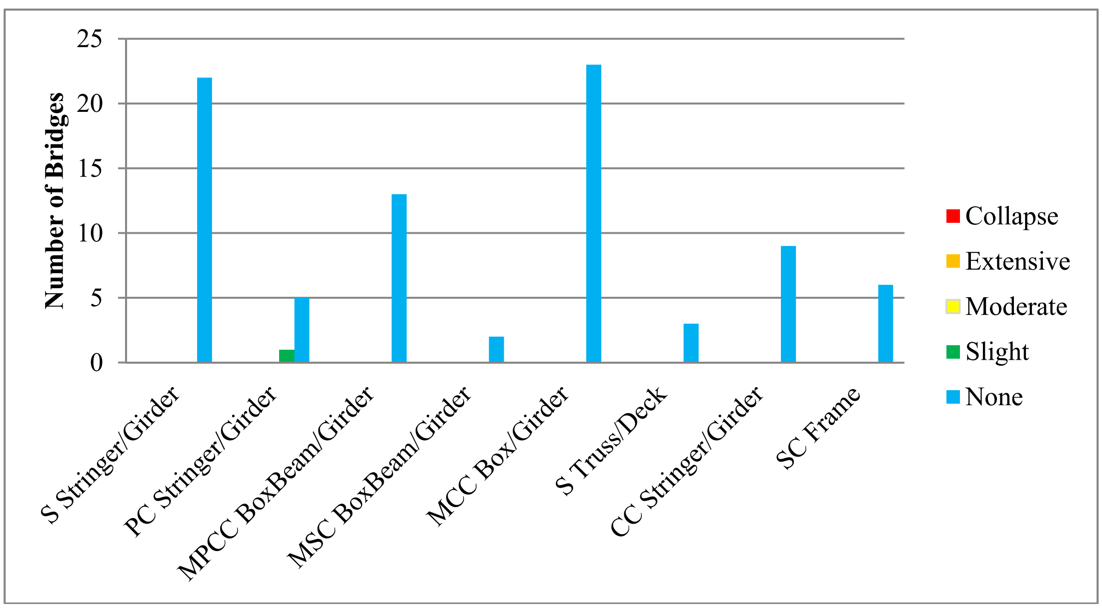

Figure C. 2: North M8.3 Scenario on I-5 from Multnomah to Clackamas County.

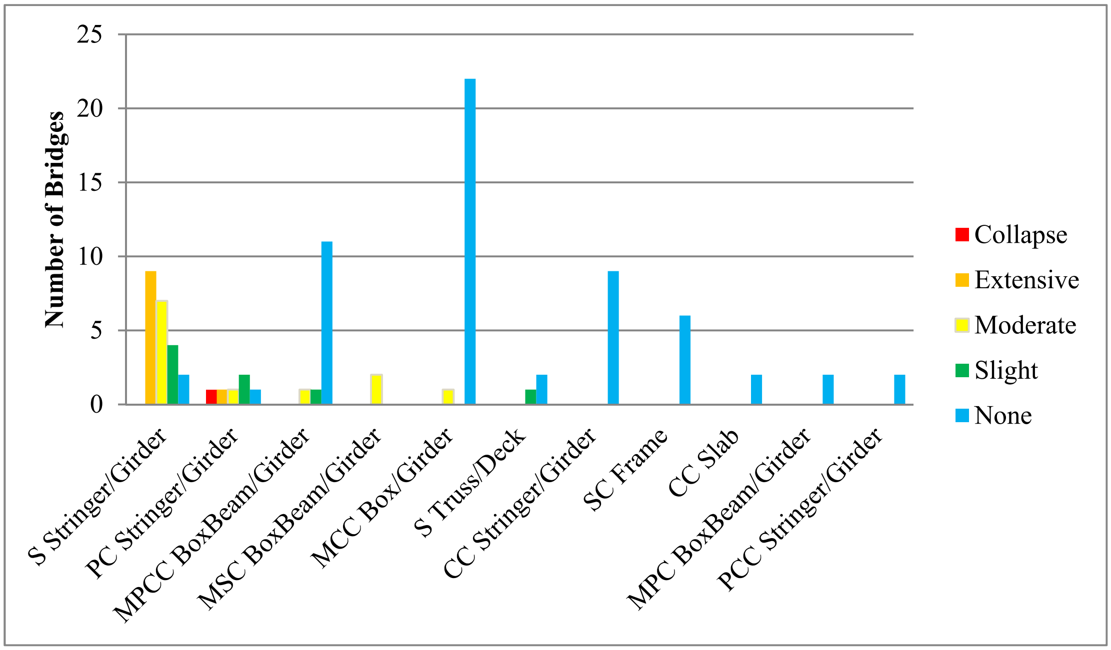

Figure C. 3: Hills M6.5 Scenario on I-5 from Multnomah to Clackamas County.

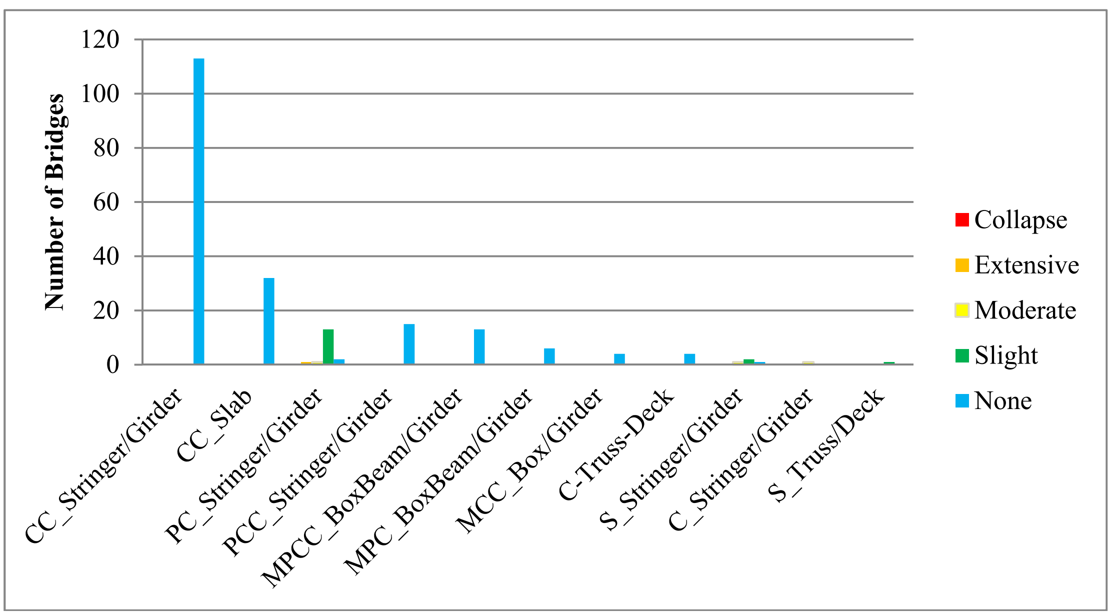

Figure C. 4: Cascadia M9.0 Scenario on I-5 from Marion to Lane County.

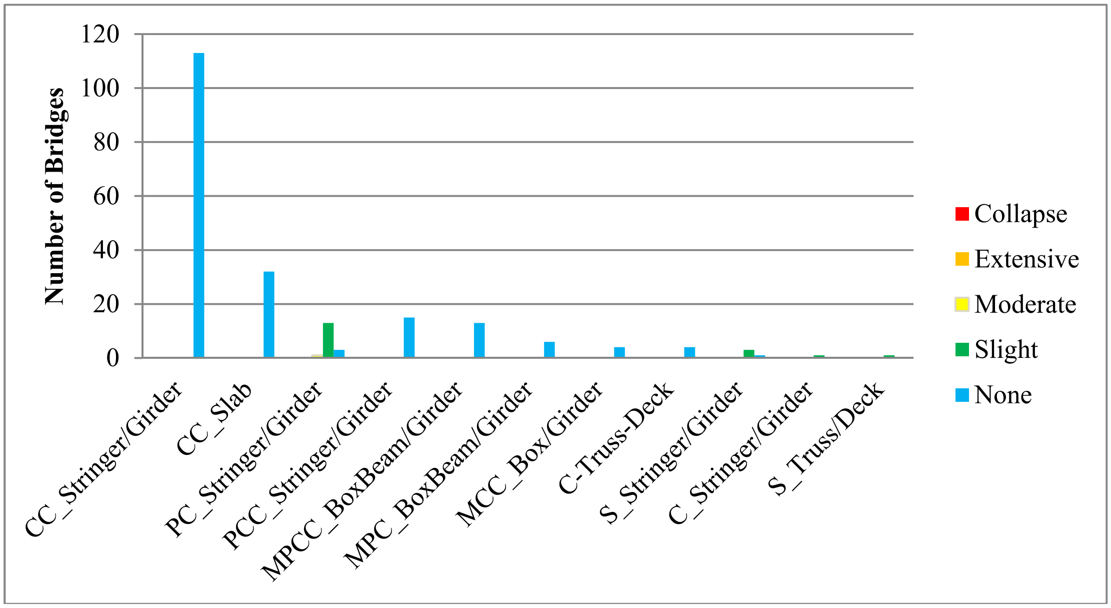

Figure C. 5: Cascadia North M8.3 Scenario on I-5 from Marion to Lane County.

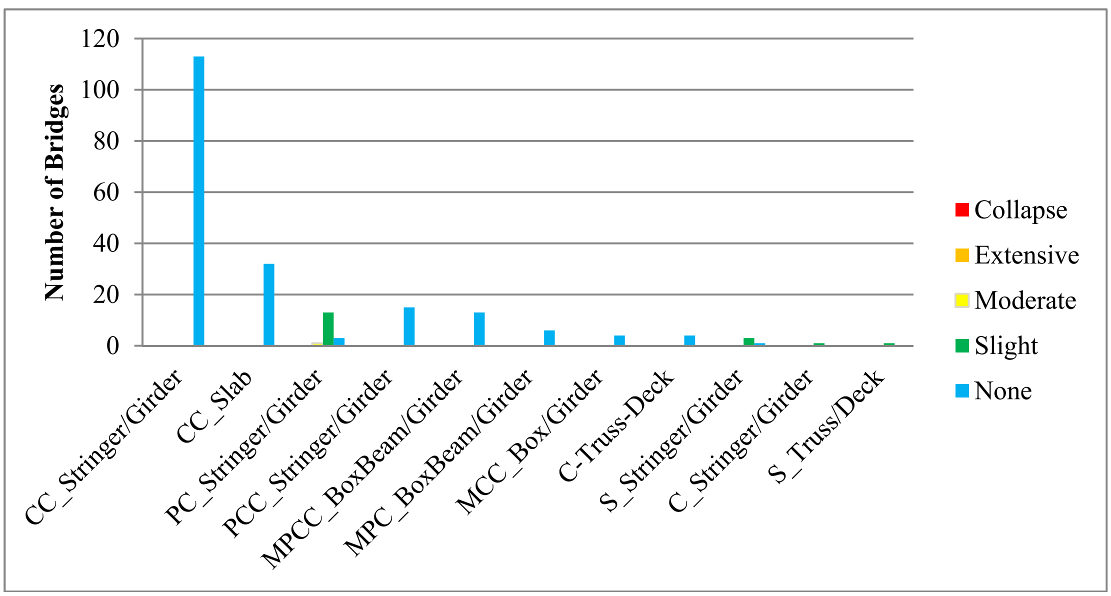

Figure C. 6: Cascadia South M8.3 Scenario on I-5 from Marion to Lane County.

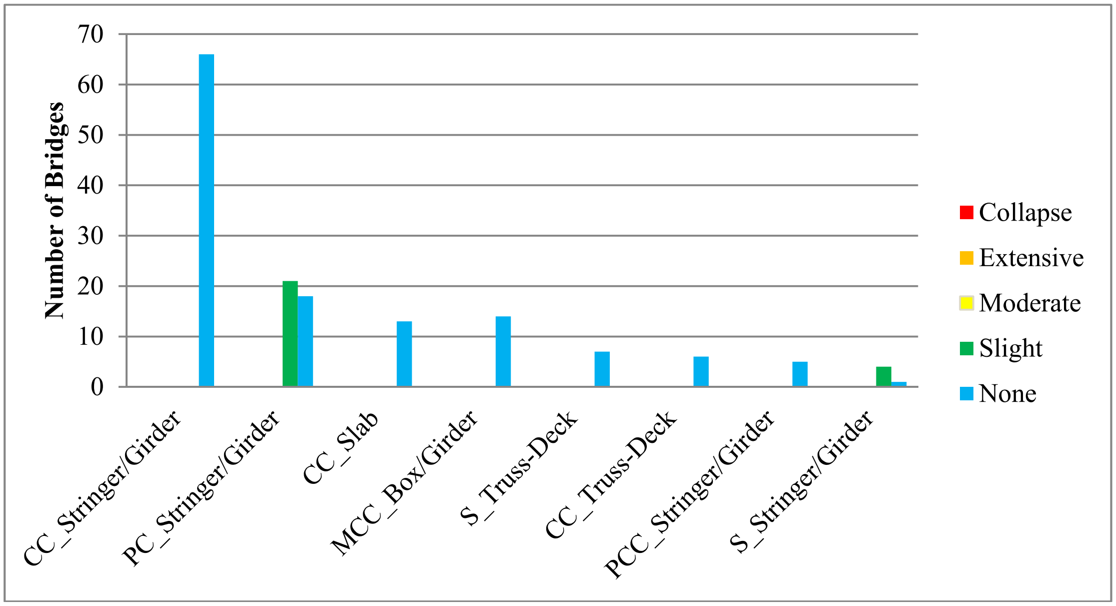

Figure C. 7: Cascadia M9.0 Scenario on I-5 from Douglas to Jackson County.

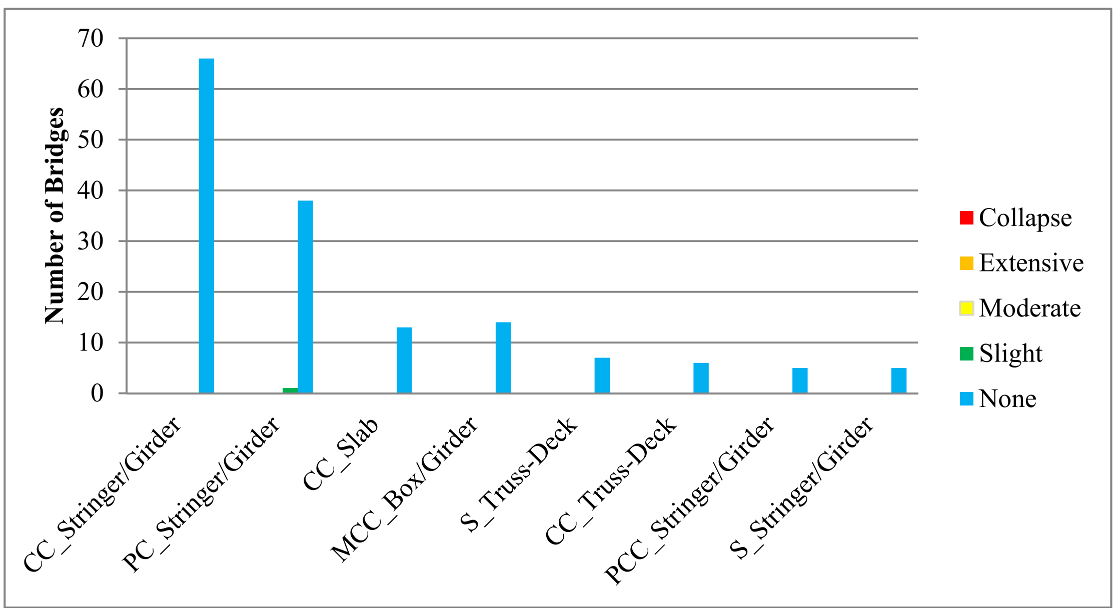

Figure C. 8: Cascadia North M8.3 Scenario on I-5 from Douglas to Jackson County.

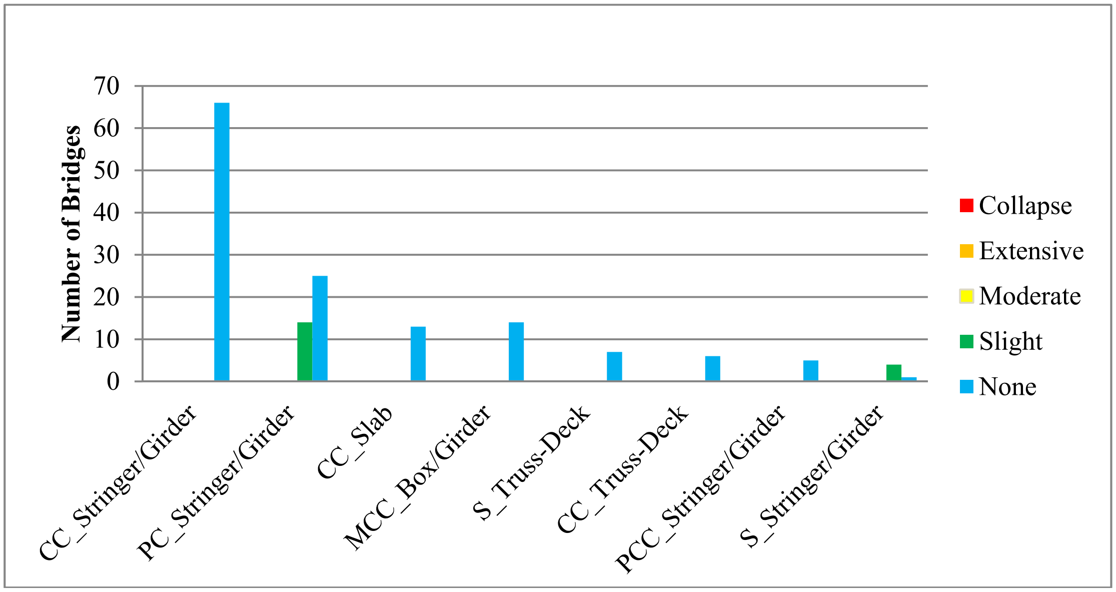

Figure C. 9: Cascadia South M8.3 Scenario on I-5 from Douglas to Jackson County.

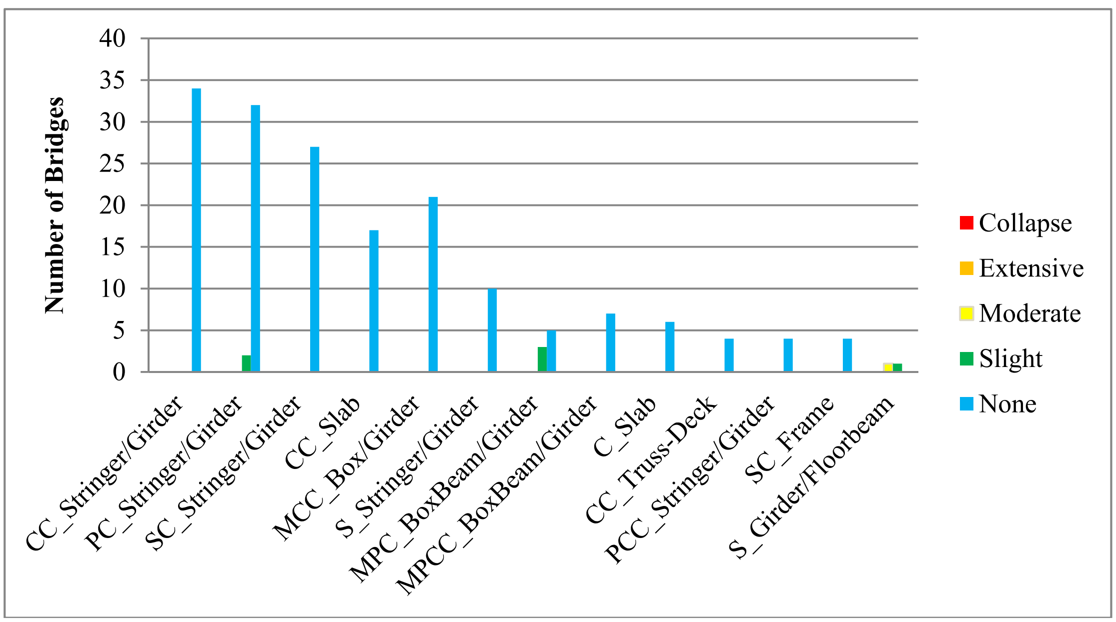

Figure C. 10: Cascadia M9.0 Scenario on I-84.

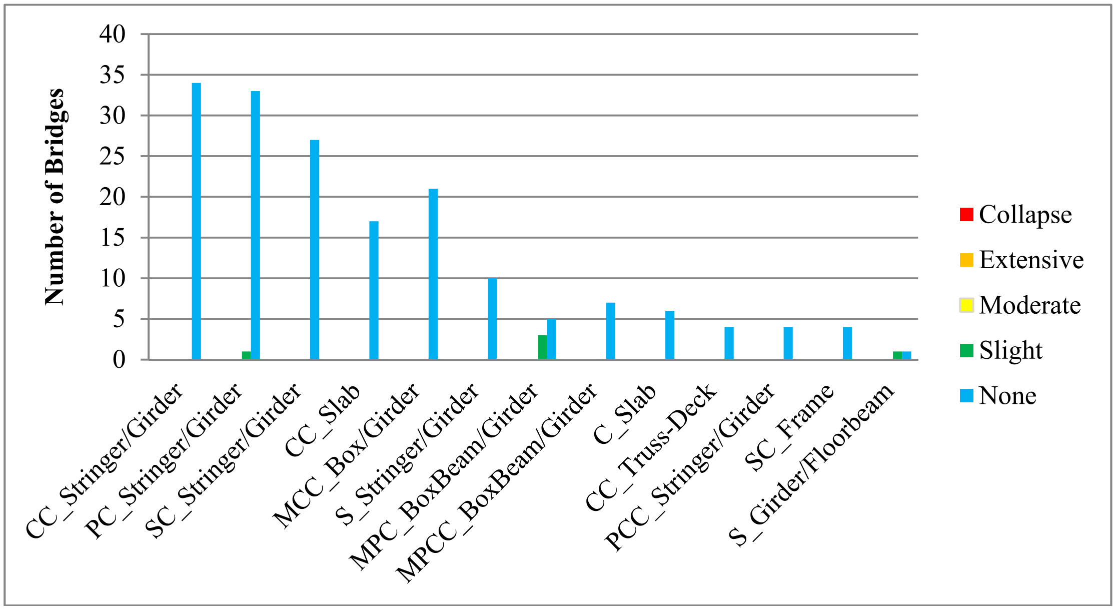

Figure C. 11: Cascadia North M8.3 Scenario on I-84.

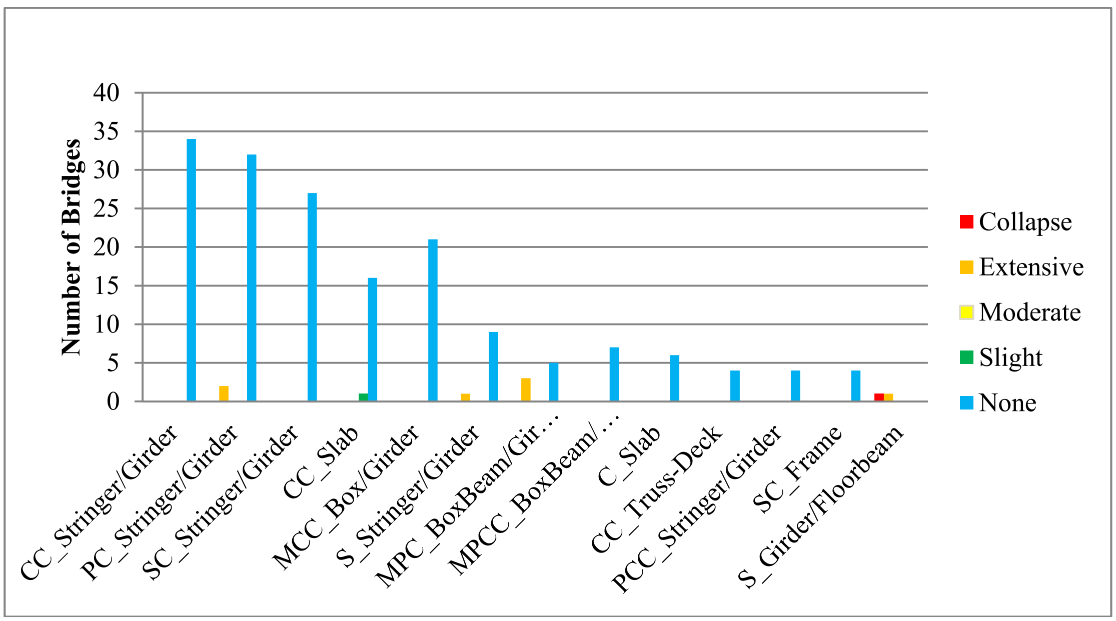

Figure C. 12: Portland Hills M6.5 Scenario on I-84.

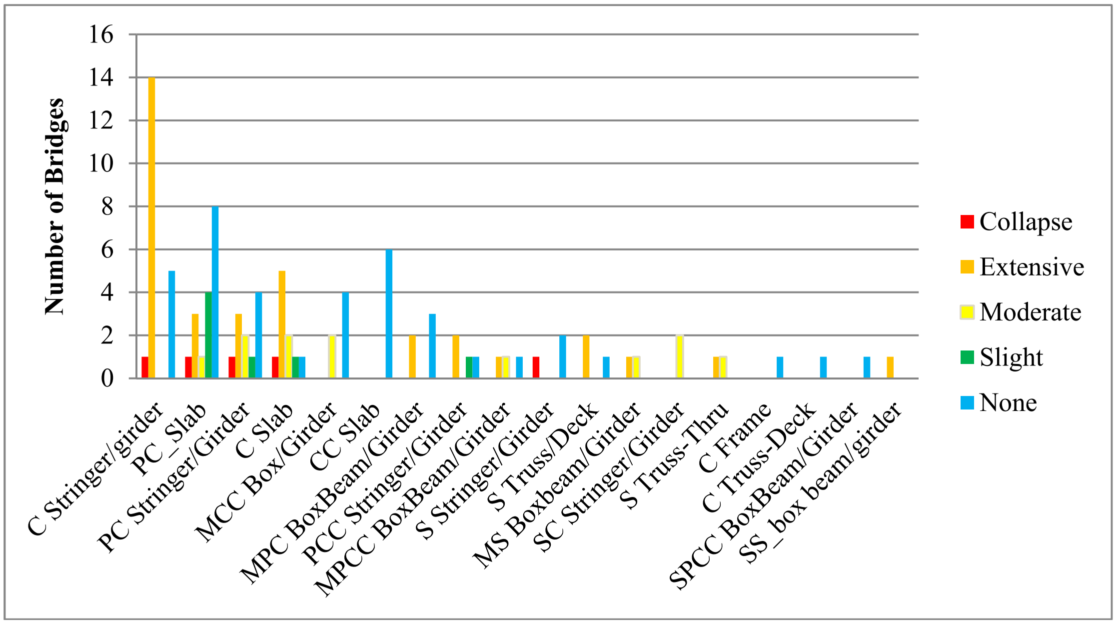

Figure C. 13: Cascadia M9.0 Scenario on US 101.

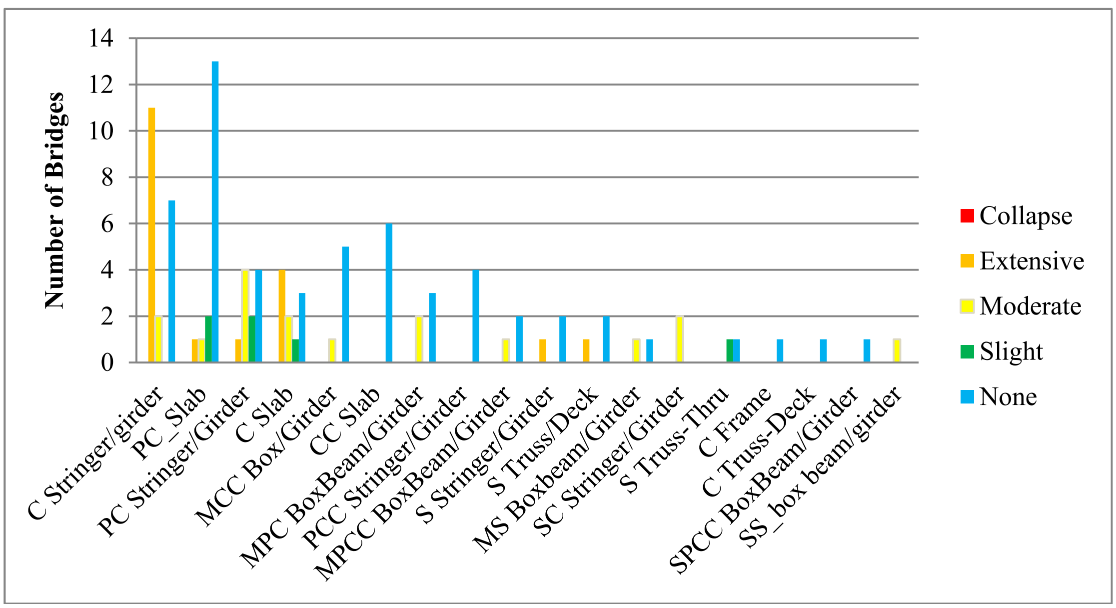

Figure C. 14: Cascadia North M8.3 Scenario on US 101

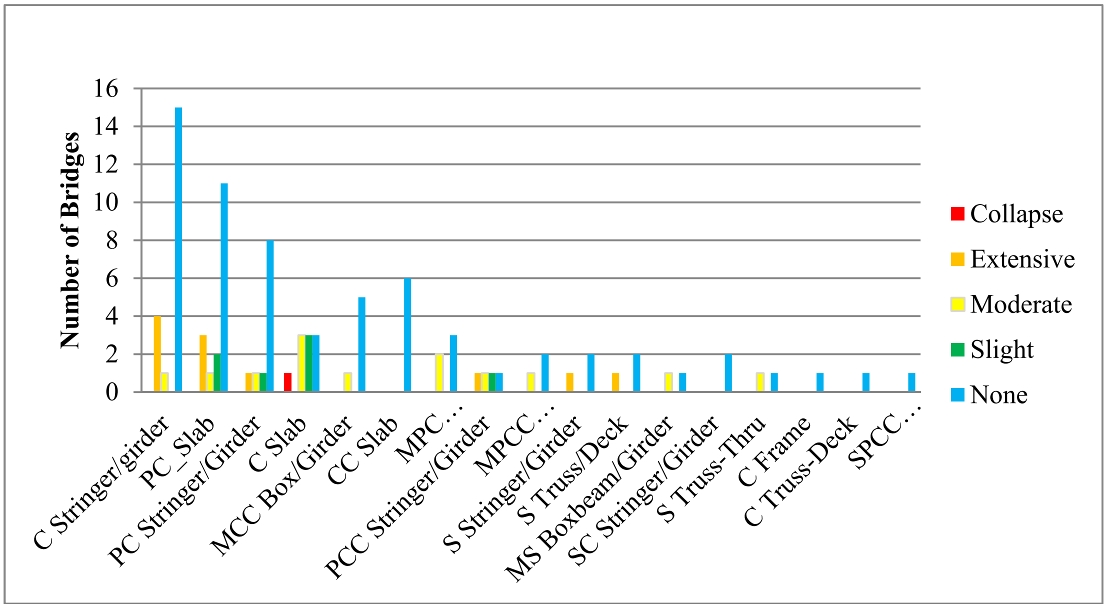

Figure C. 15: Cascadia South M8.3 Scenario on US 101.

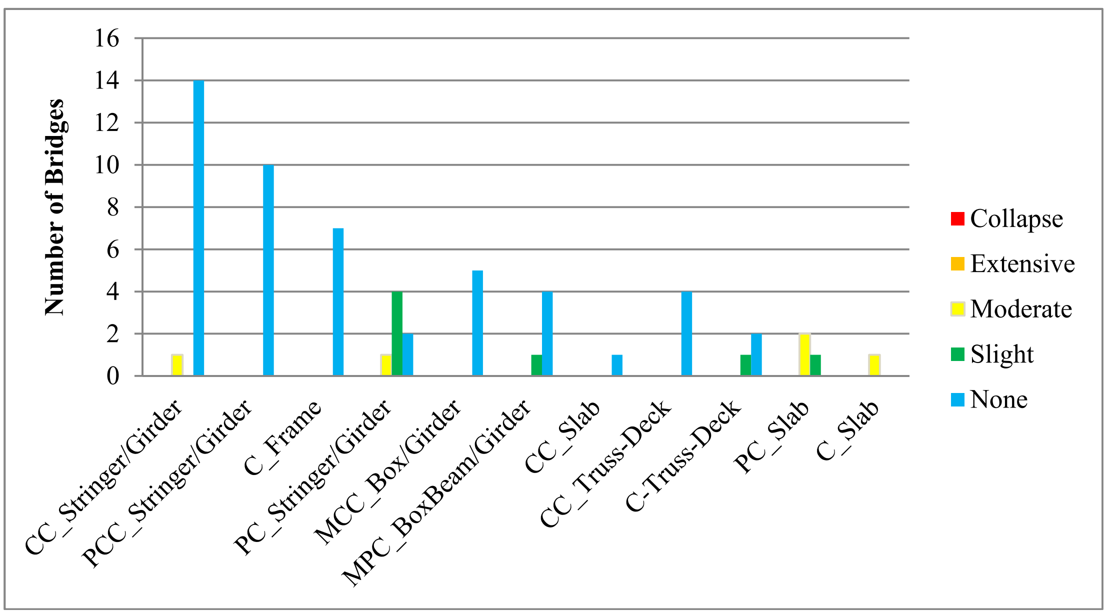

Figure C. 16: Cascadia M9.0 Scenario on US 26.

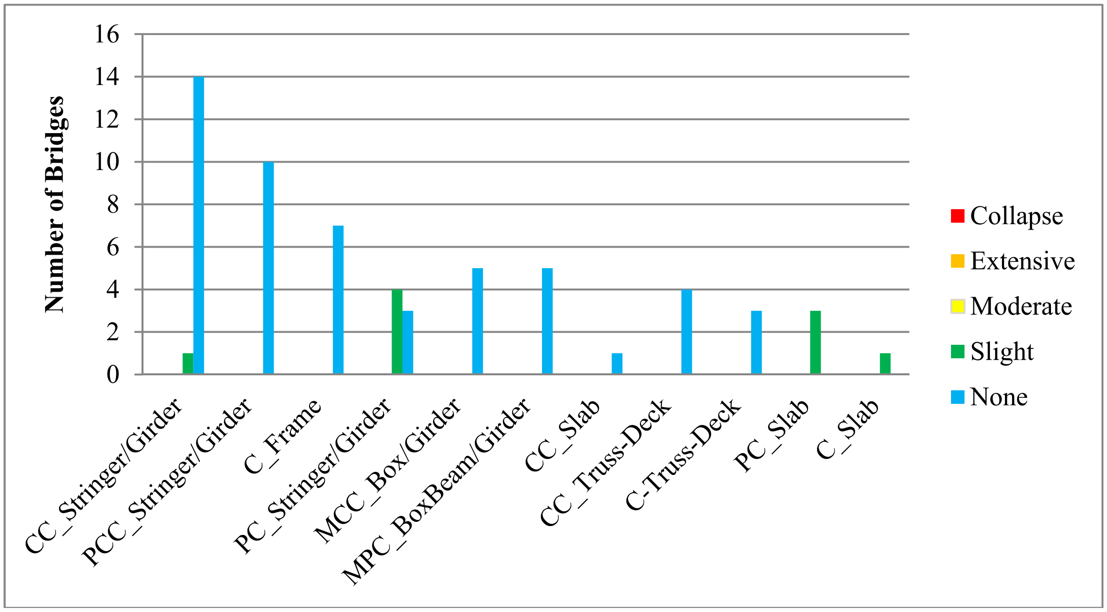

Figure C. 17: Cascadia North M8.3 Scenario on US 26.

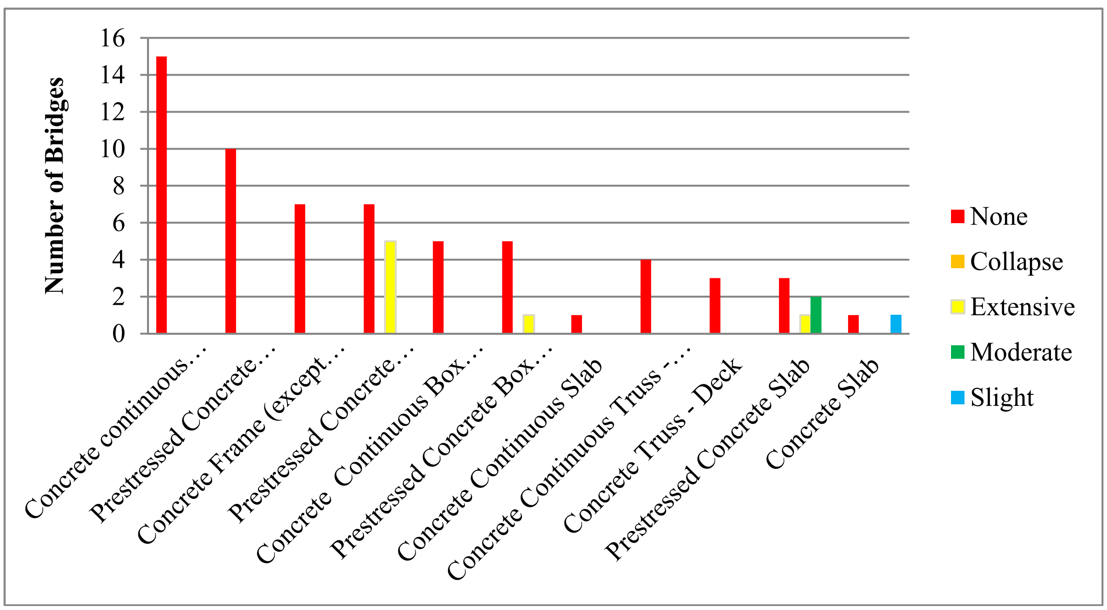

Figure C. 18: Portland Hills M6.5 Scenario on US 26.

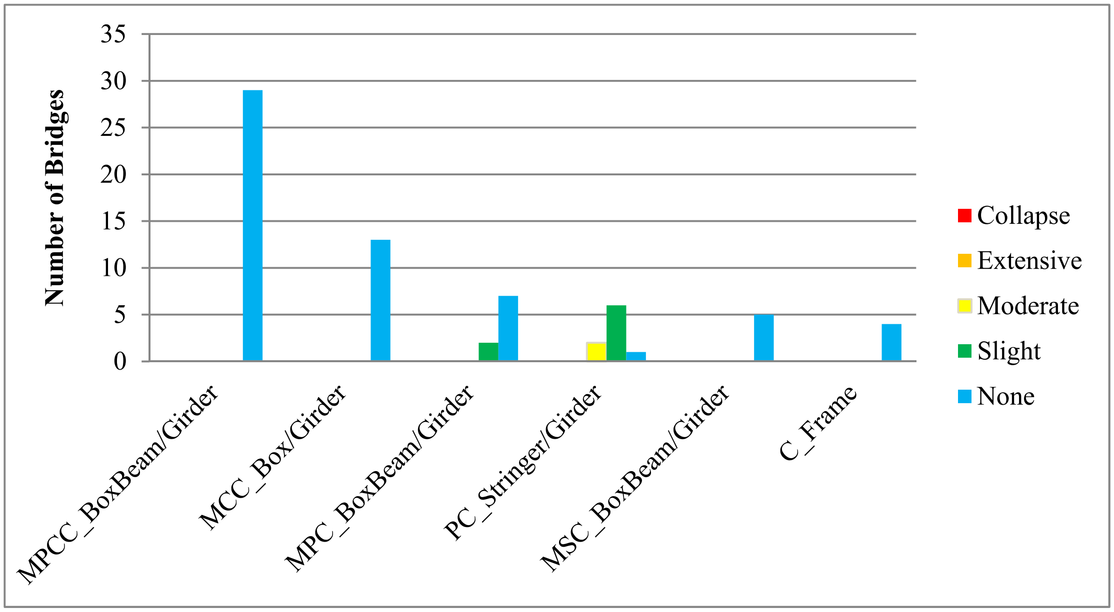

Figure C. 19: Cascadia M9.0 Scenario on I-205.

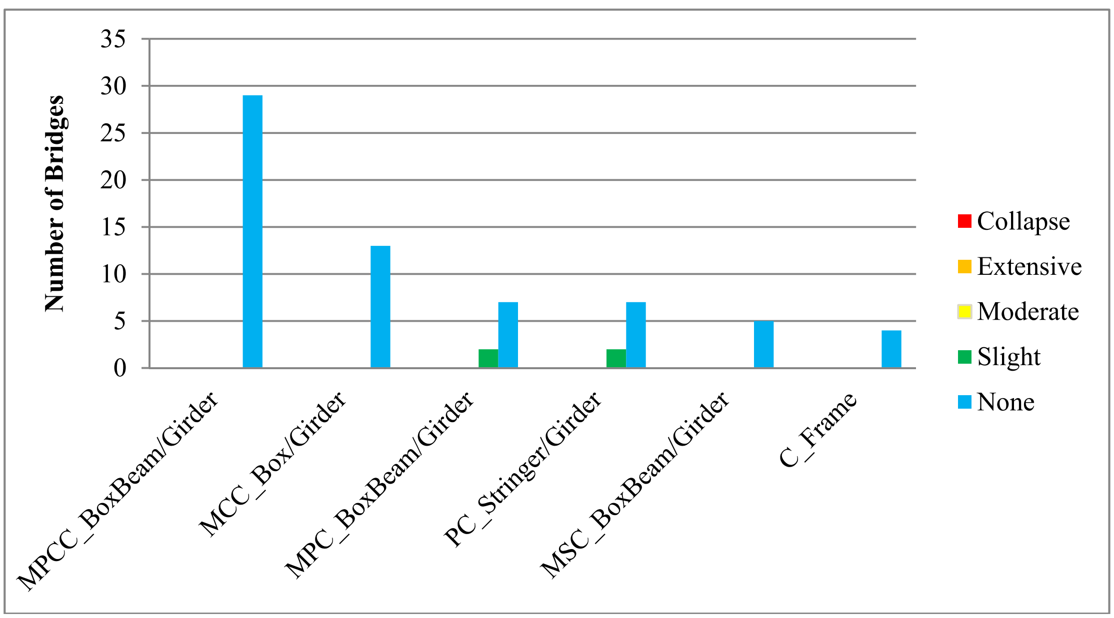

Figure C. 20: Cascadia North M8.3 Scenario on I-205.

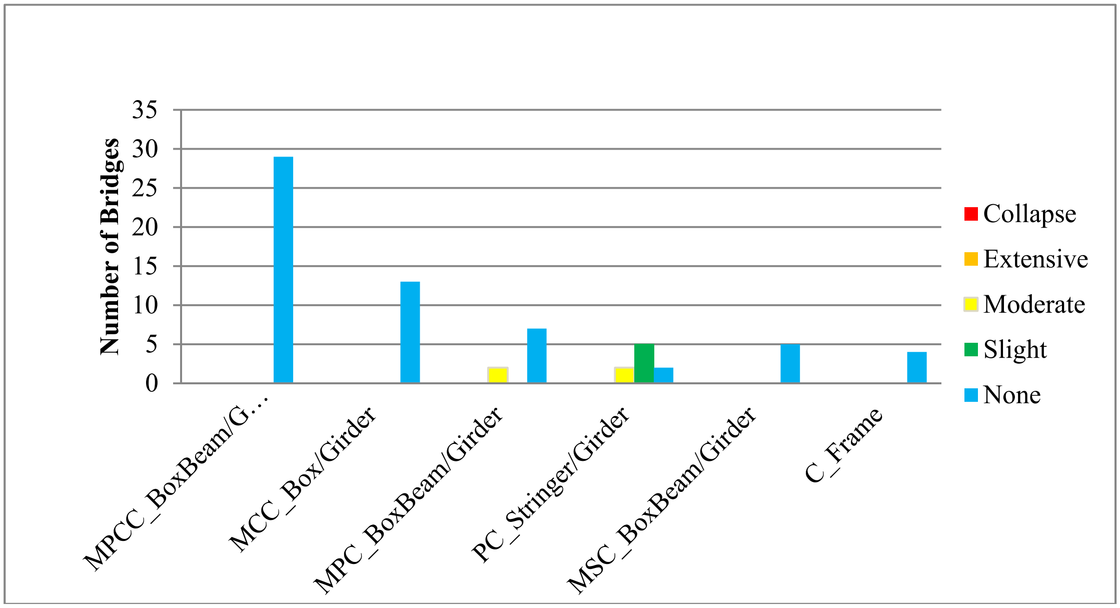

Figure C. 21: Portland Hills M6.5 Scenario on I-205.

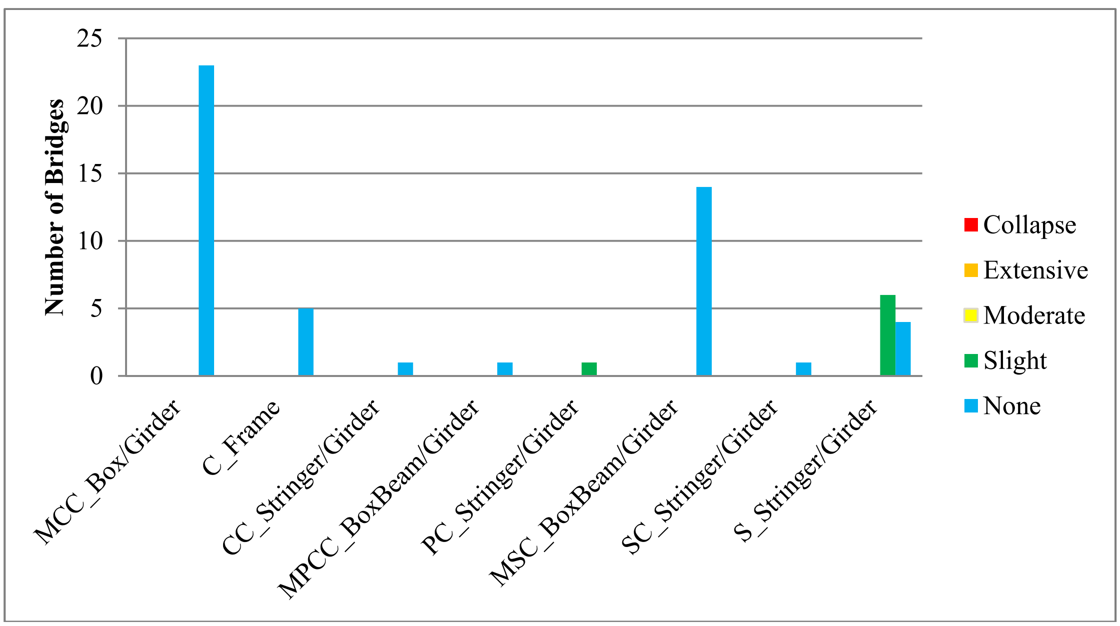

Figure C. 22: Cascadia M9.0 Scenario on I-405.

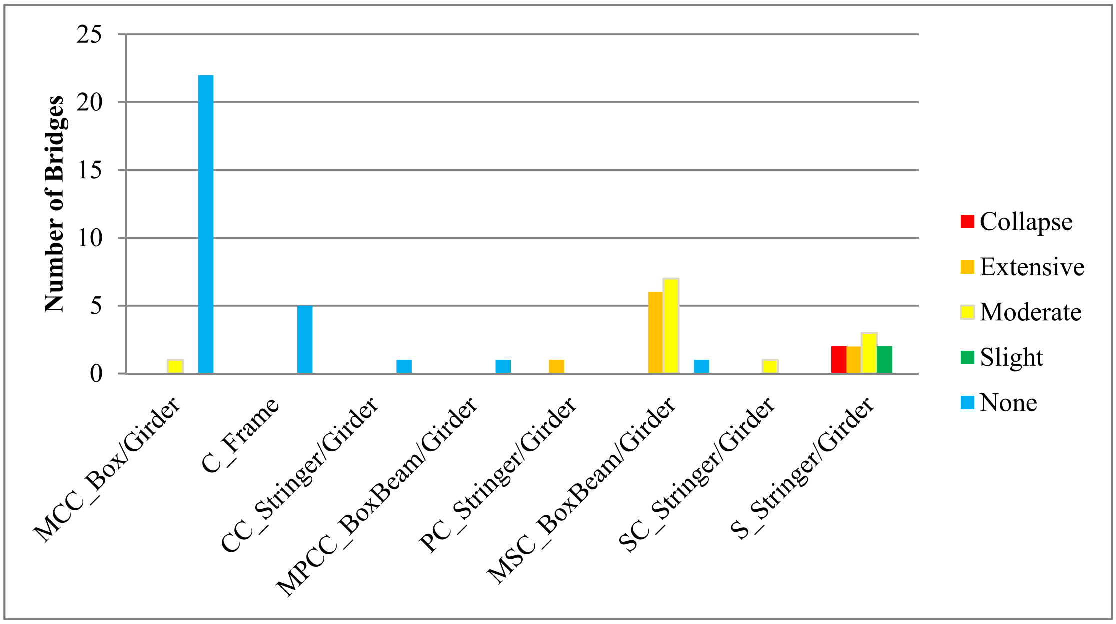

Figure C. 23: Portland Hills M6.5 Scenario on I-405.

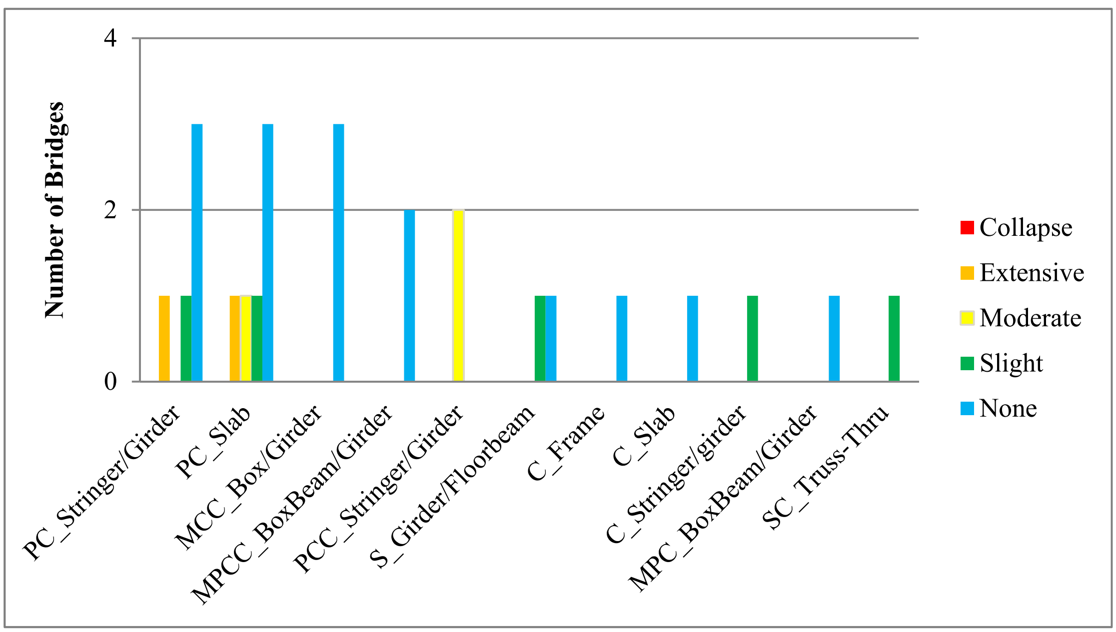

Figure C. 24: Cascadia M9.0 Scenario on US 30.

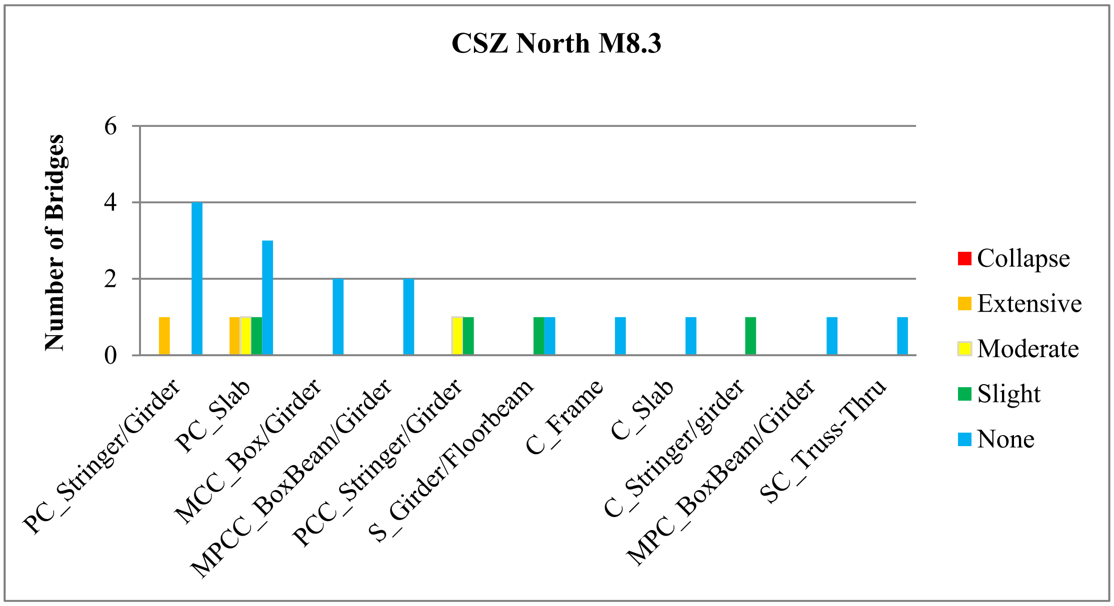

Figure C. 25: Cascadia North M8.3 Scenario on US 30.

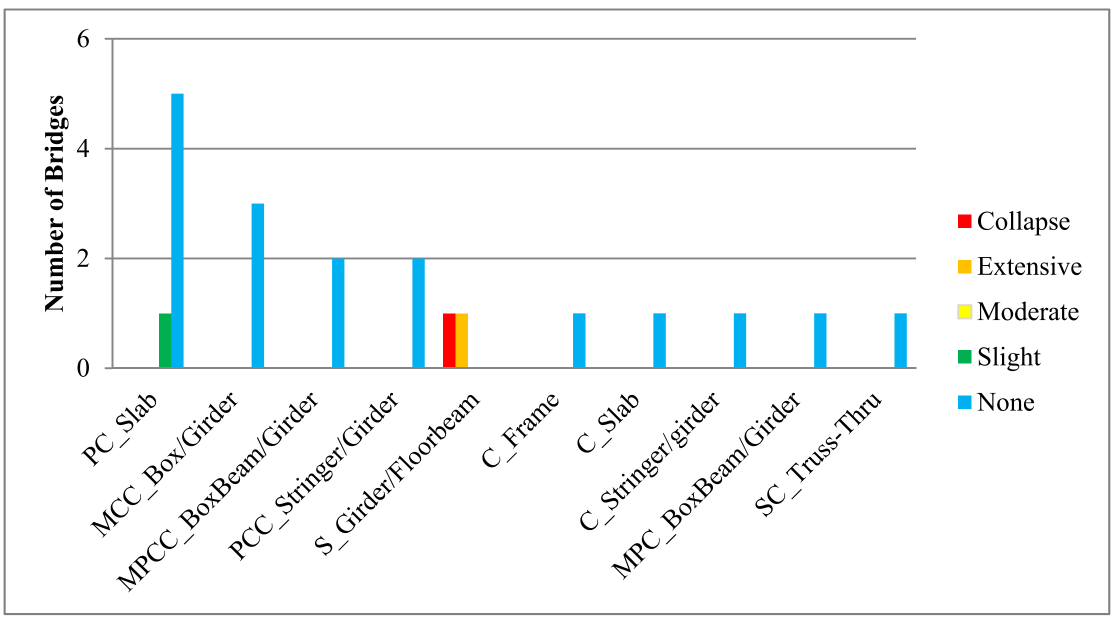

Figure C. 26: Portland Hills M6.5 Scenario on US 30.

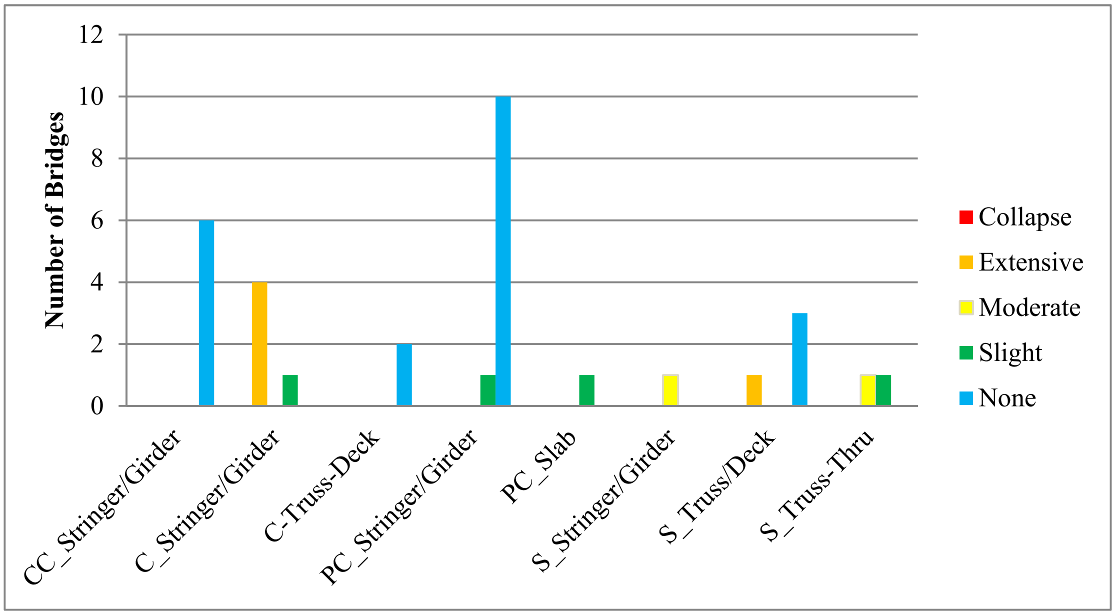

Figure C. 27: Cascadia M9.0 Scenario on US 20.

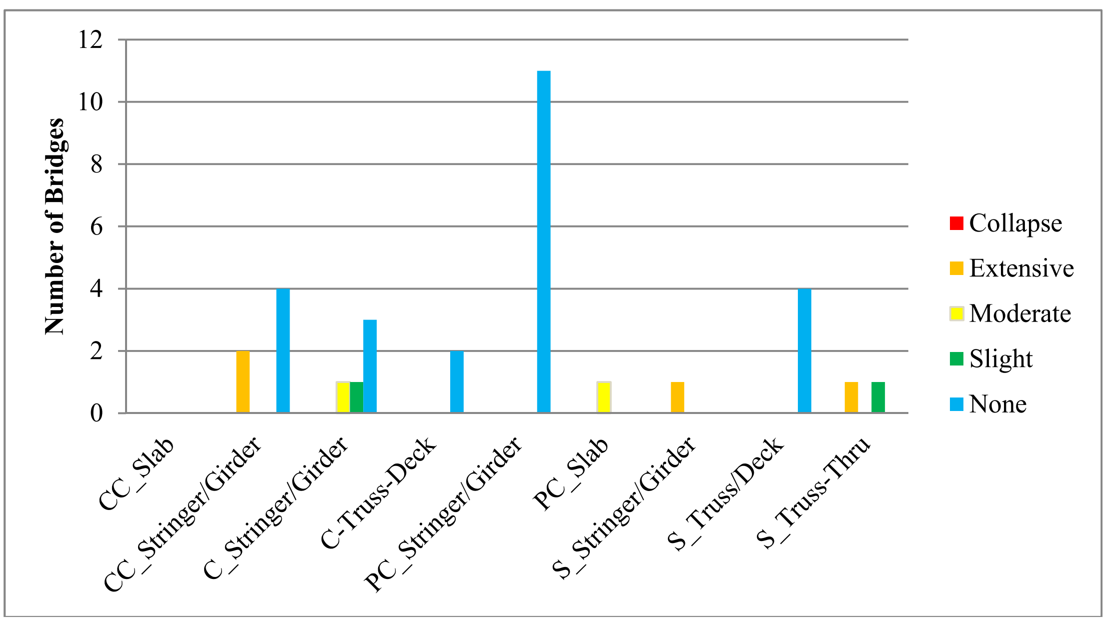

Figure C. 28: Cascadia North M8.3 Scenario on US 20.

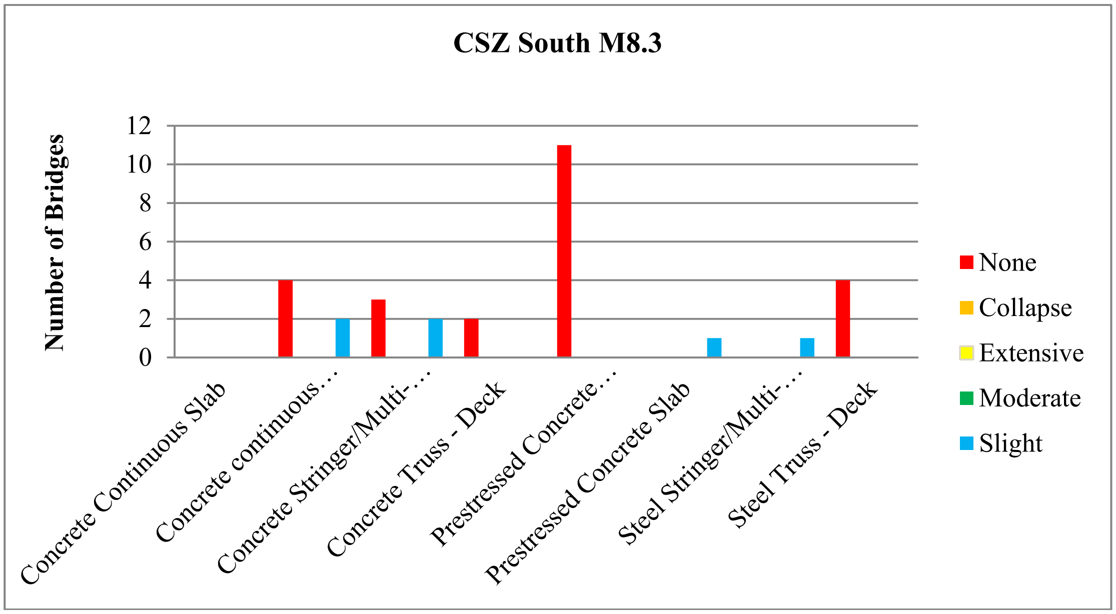

Figure C. 29: Cascadia South M8.3 Scenario on US 20.

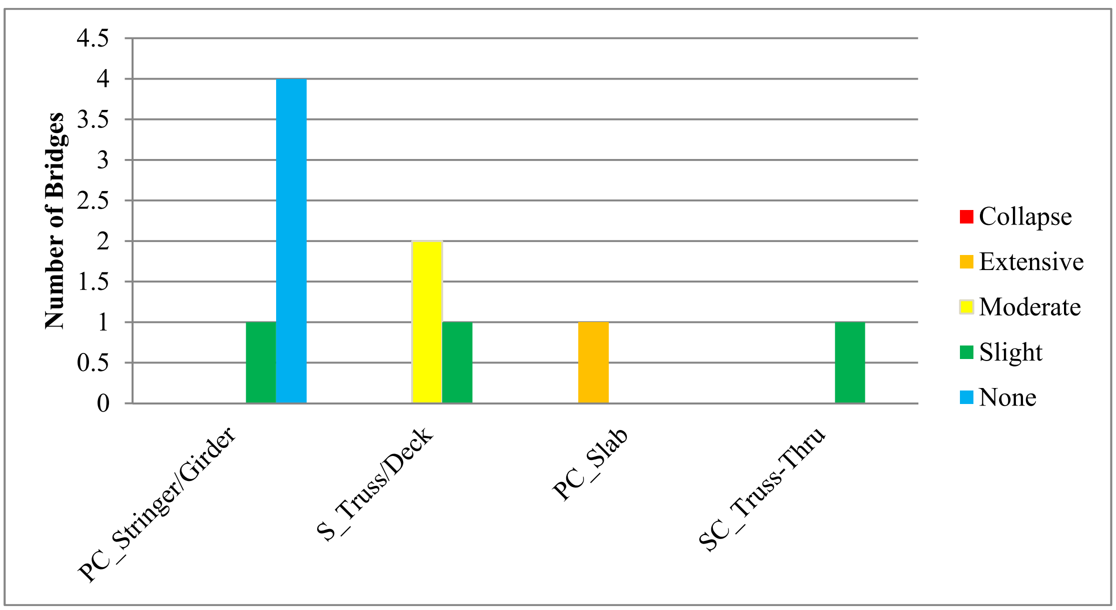

Figure C. 30: Cascadia M9.0 Scenario on OR 38.

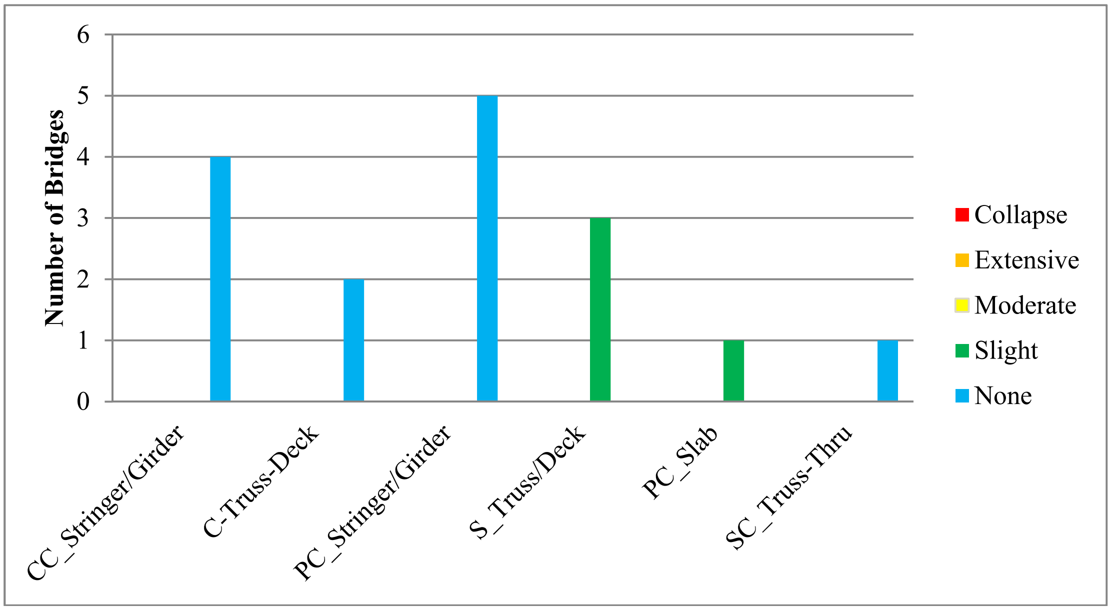

Figure C. 31: Cascadia North M8.3 Scenario on OR 38.

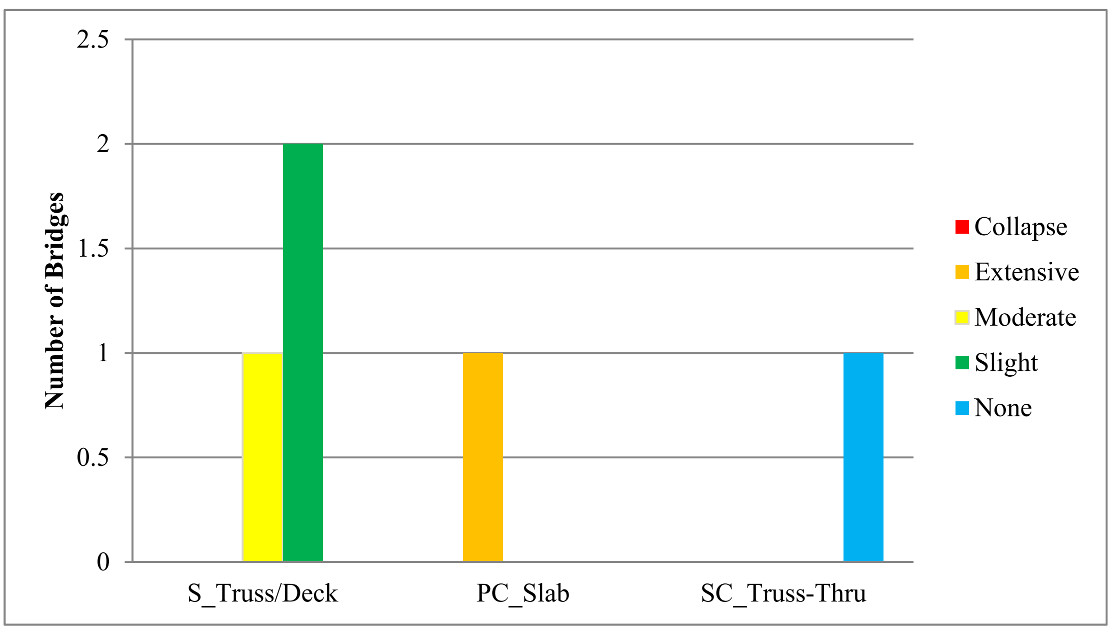

Figure C. 32: Cascadia South M8.3 Scenario on OR 38.

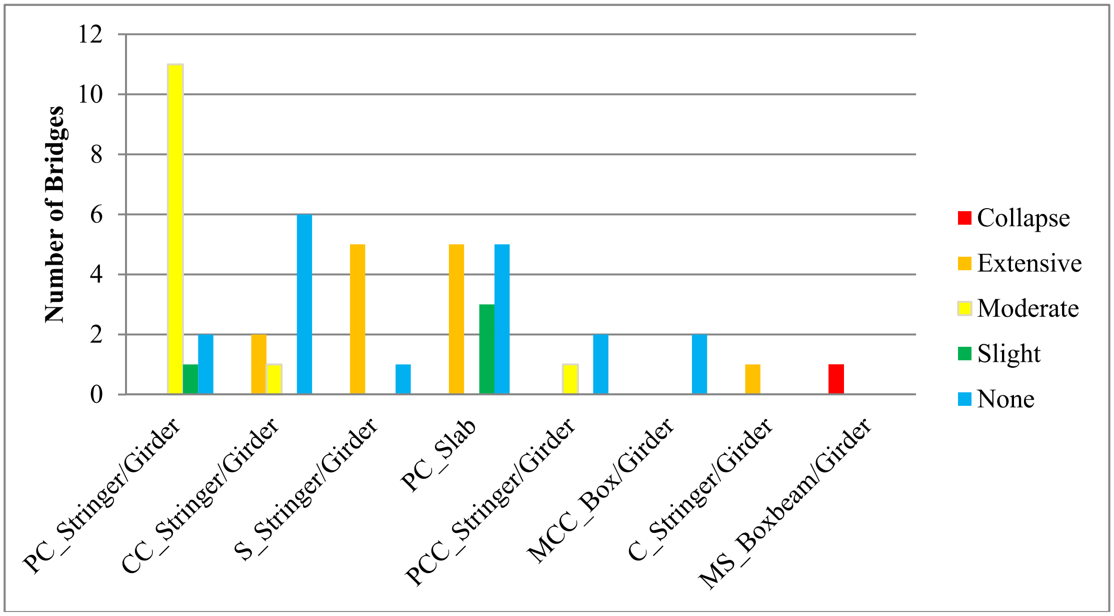

Figure C. 33: Cascadia M9.0 Scenario on OR 42.

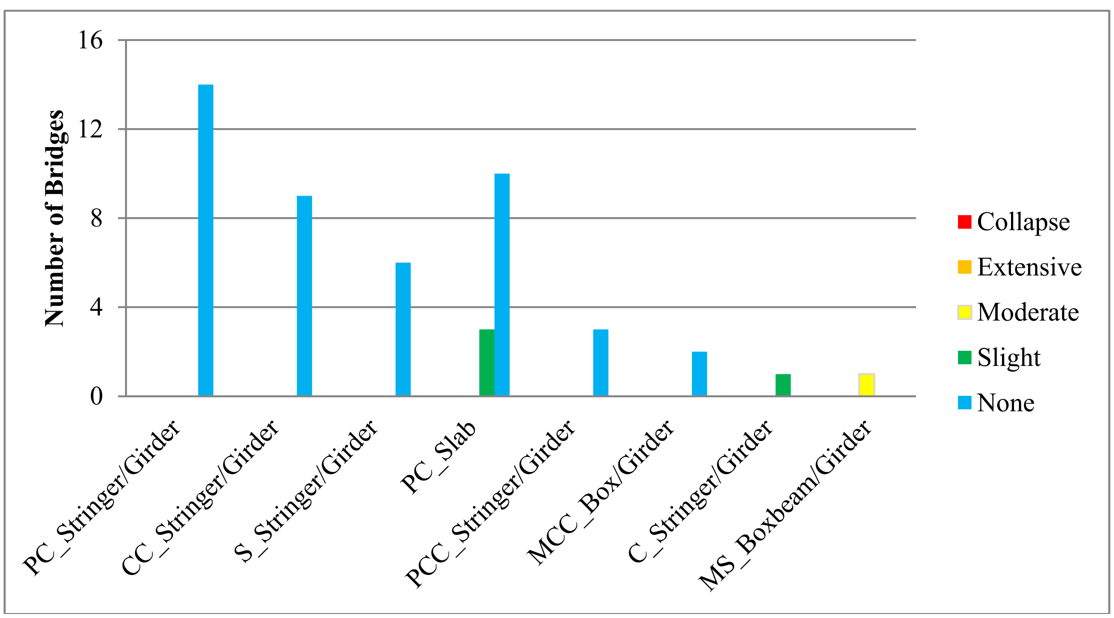

Figure C. 34: Cascadia North M8.3 Scenario on OR 42.

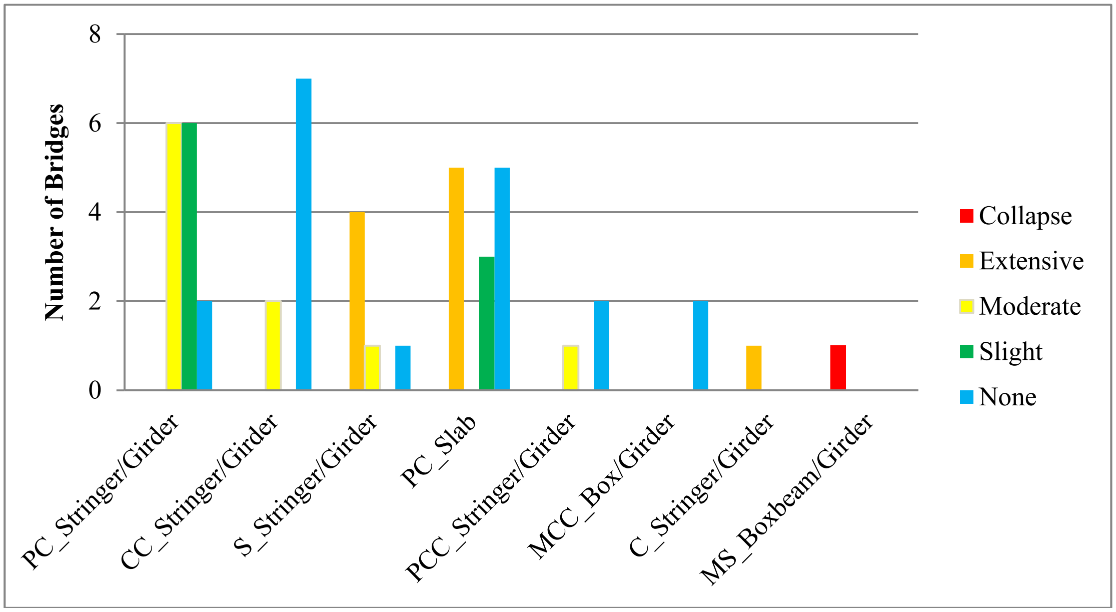

Figure C. 35: Cascadia South M8.3 Scenario on OR 42.

11.4 APPENDIX D - BRIDGE CLASSIFICATION BY MATERIAL AND DESIGN PER ROUTE

Table D. 1: Bridge Classification by Material and Design per Route.

Material and Design	I-5 MWC	I-5 MLL	I-5 DJJ	I 84	US 101	US 26	I 205	I 405	US 30	US 20	OR 38	OR 42
Concrete continuous Stringer/Multi-beam or Girder	9	113	66	34	42	15	0	1	9	6	4	9
Prestressed Concrete Stringer/Multi-beam or Girder	6	17	39	34	11	7	9	1	5	11	5	14
Concrete Continuous Box Beam or Girders - Multiple	23	4	14	21	6	5	13	23	3	0	2	2
Concrete Continuous Slab	2	32	13	17	6	1	0	0	4	0	0	1
Prestressed Concrete Continuous Box Beam or Girders - Multiple	13	13	3	7	3	1	29	1	2	0	0	0
Steel Stringer/Multi-beam or Girder	22	4	5	10	3	2	3	10	0	1	0	6
Prestressed Concrete Slab	1	2	8	7	17	3	0	0	6	1	1	13
Prestressed Concrete Continuous Stringer/Multi-beam or Girder	2	15	5	4	4	10	2	0	2	0	0	3
Prestressed Concrete Box Beam or Girders - Multiple	2	6	2	8	5	5	9	0	1	0	0	1
Steel Continuous Stringer/Multi-beam or Girder	2	1	3	27	2	2	1	1	0	0	0	0
Concrete Stringer/Multi-beam or Girder	1	1	0	0	20	1	0	0	1	5	0	1
Concrete Slab	0	1	3	6	10	1	2	0	1	0	0	1
Steel Truss - Deck	3	1	7	3	3	2	0	0	0	4	0	1
Concrete Frame (except frame culverts)	1	1	0	3	1	7	4	5	1	0	0	0
Steel Continuous Box Beam or Girders - Multiple	2	0	0	0	0	0	5	14	0	0	0	0
Concrete Continuous Truss - Deck	0	2	6	4	0	4	1	0	0	0	0	0
Material and Design	**I-5 MWC**	**I-5 MLL**	**I-5 DJJ**	**I 84**	**US 101**	**US 26**	**I 205**	**I 405**	**US 30**	**US 20**	**OR 38**	**OR 42**
Concrete Truss - Deck	0	4	3	1	1	3	0	0	0	2	2	0
Steel Continuous Frame	6	0	0	4	1	0	0	0	0	0	0	0

(except frame culverts)												
Prestressed Concrete Continuous Box Beam or Girders - Single or Spread	1	2	0	0	1	0	3	0	0	0	0	0
Steel Truss - Thru	0	0	1	1	2	0	0	0	0	2	0	0
Steel Continuous Girder and Floor beam System	0	0	2	3	0	0	0	0	0	0	0	0
Steel Girder and Floor beam System	1	0	0	2	0	0	0	0	2	0	0	0
Prestressed Concrete Continuous Slab	0	1	1	1	0	0	0	0	0	0	0	1
Concrete Continuous Box Beam or Girders - Single or Spread	0	0	1	0	0	1	1	0	0	0	0	0
Prestressed Concrete Box Beam or Girders - Single or Spread	0	1	0	1	0	1	0	0	0	0	0	0
Steel Box Beam or Girders - Multiple	0	0	0	0	2	0	0	0	0	0	0	1
Steel Continuous Truss - Deck	0	0	0	0	1	2	0	0	0	0	0	0
Steel Continuous Truss - Thru	0	0	0	0	1	0	0	0	1	0	1	0
Prestressed Concrete Tee Beam	0	0	1	1	0	0	0	0	0	0	0	0
Steel Box Beam or Girders - Single or Spread	0	0	0	0	1	1	0	0	0	0	0	0
Concrete Box Beam or Girders - Multiple	0	0	1	0	0	0	0	0	0	0	0	0
Concrete continuous Girder and Floor beam System	0	0	0	0	0	1	0	0	0	0	0	0
Concrete Continuous Tee Beam	0	0	0	0	0	1	0	0	0	0	0	0
Prestressed Concrete Continuous Girder and Floor beam System	0	0	0	1	0	0	0	0	0	0	0	0

11.5 APPENDIX E - SKEW ANGLE INCONSISTENCY

Table E. 1: Skew angle definition inconsistent with REDARS2.

NBI Ref	Name	NBI Skew Angle
13514H002 00660	I-84 (HWY 002) over CONN 1 (HWY 2) at 2.0 MI E OF NE 74TH AVE	60
16098 072 00173	HWY 72 over CLAGGET CR at 0073 N OF SALEM CITY L	78
09883 064 00969	I-205 (HWY 064) over CREEK at 0.4 M E OF 1E INTERCHANGE	59
18408 002 07116	I-84 (HWY 002) over UPRR at 111 MI W THE DALLES	59
09672 143 00920	OR 210 (HWY 143) over HWY 144 at 1.5 MI SW BEAVERTON CC	55
09702 064 00951	I-205 (HWY 064) over MAIN ST at 0.4 MI N OF W. LINN BR	55
09718 171C00409	OR 224 (HWY 171)CO over HWY 64 at 2.9 MI N OF CLACKAMAS RV	55
19199 047 07083	US 26 (HWY 047) over HWY 29 CANYON RD at 3.7 MI W OF PORTLAND CC	55
01205A033 04568	US 20 (HWY 033) over WPRR & HARRIS RD at 05.0 MI W OF PHILOMATH	54
07794B001 28228	I-5 (HWY 001) SB over HWY 51 at 020 MI N MARION-CLACK LN	54
07832 001 17475	I-5 (HWY 001) CON over CENTRAL ORE RR at 005 MI S OF COTTAGE GROVE	54
18268 162 00142	OR 22 (HWY 162) over I-5 (HWY 001) at INTER.N SANTIAM HWY ANDI5	54
01868 037 00578	OR 6 (HWY 037) over WILSON RIVER MILLS BRDG. at 058 MI E OF HWY 9 JCT	53
09717 064 01376	I-205 (HWY 064) NB over UPRR at 4.6 MI N OF OREGON CITY	52
09717A064 01376	I-205 (HWY 064) SB over UPRR at 4.6 MI N OF OREGON CITY	52
07800A001 27469	BROADACRES RD over I-5 (HWY 001) at 028 MI N ORE 214	51
13540 064 01743	I-205 (HWY 064) NB over PEDESTRIAN PATH at 0.8 MI N OF PORTLAND SCL	51
13540A064 01743	I-205 (HWY 064) SB over PEDESTRIAN PATH at 0.8 MI N OF PORTLAND SCL	51
19107 001 08652	I-5 (HWY 001) SB over COW CREEK at I5 NB MP 86.52	51
07715 02W 06082	HWY 2W over SWEDETOWN COUNTY R at 1.0 MI E CLATSKANIE	50
07728A001C29130	UPPER BOONES FERRY over I-5 (HWY 001) at 1.5 MI N OF TUALATIN RV	49
07854C001 25910	I-5 (HWY 001) over UPRR at 003 MI S OF SALEM NCL	49
08939 062 03240	OR 126 (HWY 062) over CHICHAHOMINY CREEK at 003 MI W WALTON	49
18480 001C29225	HWY 1NB TO HWY 144 over I-5 (HWY 001) at 2.63 MI N OF TUALATIN RV	49
01601 045 03876	OR 38 (HWY 045) over ELK CR 2ND XING at 029 MI E ELKTON	48
08341 001 05578	I-5 (HWY 001) over HWY 025 SPUR at 25.3 MI N OF MEDFORD NCL	48
08605 002 04505	I-84 (HWY 002) EB over HWY 100 at IN CASCADE LOCKS	48

Table E. 2: No drawing available.

NBI Ref	Name	NBI Skew Angle
03172A035 00414	OR 42 (HWY 035) over CENTRAL ORE RR at 06.0 MI W COQUILLE	57
03849B020 00125	Murdock Creek, Hwy 19	90
18142 270 05883	_	90
17459 021 00250	OR 66 (HWY 021) over NEIL CREEK at 01.2 MI E OF HWY 001	79
00598D244 01678	OR 42S (HWY 244) over COQUILLE RIVER at 00.5 W JCT HWY 035	75
07369 002 19762	US 730 (HWY 002) over UPRR at 5.7 MI W WASHINGTON ST LN	74
07110 018 00808	OR 58 (HWY 018) over UPRR at 081 MI E HWY 1 JCT	65
02625A191 03072	OR 223 (HWY 191) over MARYS RIVER at 007 MI N WREN	63
02020A004 27995	USRS Canal J, Hwy 426	60
20337 001 13200	I-5 (HWY 001) NB over I-5 @ WILBUR-UMPQUA RD - at I-5 (HWY 001) NB	60
02474B004 25252	Link River, Hwy 20	58
07770 162C00191	LANCASTER over OR 22 (HWY 162) CONN at 012 MI E OF SALEM	57
09635 006 26492	I-84 (HWY 006) WB over UNION JCT INTCH WB at 3.1 MI. E. JCT. OR 82	57
09635A006 26492	I-84 (HWY 006) EB over UNION JCT INTCH EB at 3.1 MI. E. JCT. OR 82	57
00406A021 00076	OR 66 (HWY 021) over CENTRAL ORE RR at 006 MI W HWY I	56
17460 021 00317	OR 66 (HWY 021) over NEIL CREEK at 01.8 MI E OF HWY 001	56
18787 019 10927	Little Deschutes River, Hwy 18	56
07817 162 04289	OR 22 (HWY 162) over SLIDE VIADUCT at 069 MI W DETROIT	55
18276 050 00157	_	55
08073 162 00544	OR 22 (HWY 162) over JOSEPH ST at 036 MI E SALEM	54
19183 162 06642	OR 22 (HWY 162) over MARION CREEK at IN MARION FORKS	53
00776A052 04547	OR 74 (HWY 052) over HINTON CREEK(HEPPNER) at IN HEPPNER	50
17424 162 00890	OR 22 (HWY 162) WB over BEAVER CREEK at 6.1 MI E SALEM	50
20212 001 13046	I-5 (HWY 001) SB over I-5 @ CORP & CO RD at 4.3 MI S OF SUTHERLIN SCL	50
470415000 00000	13TH ST EAST over SHELTON DITCH at PARALLEL TO 470414	50
02147 020 01839	Link River & Hwy 20, Hwy 4	49
07171 018 03709	OR 58 (HWY 018) over PRIVATE LOGGING ROAD at 020 MI E OAKRIDGE	49
08981 030 02486	OR 22 (HWY 030) over OR 221 (HWY 150) CONN at 001 MI E SALEM	49
00646A026 09749	OR 35 (HWY 026) over MHR & UNNAMED CR at 5.0 MI S HOOD RIVER ECL	48
02010 01W 00486	OR 99W (HWY 001W) over SW MULTNOMAH BLVD at 1.7 MI S OF HWY 40	47
08614 300 00551	OR 206 (HWY 300) over GRASS VALLEY CANYONCREEK at 055 MI SE WASCO	46
18153 002 08213	RIVER RD over I-84 (HWY 002) at WEST THE DALLES	58
08232N001 22242	I-5 (HWY 001) NB over BUTTE CREEK at 191 MI N LANE-LINN LINE	55
00860A033 01525	US 20 (HWY 033) over SIMPSON CREEK at 082 MILE E. OF TOLEDO	50
09015A001 30686	I-5 (HWY 001) over HWY 1W NB TO HWY 1 NB	65

Table E. 3: Skew angle definition consistent with REDARS2.

NBI Ref	Name	NBI Skew Angle
02363 047 04947	US 26 (HWY 047) over HWY 102 DAVIES/PTB at 4.0 MI W HWY 37 JCT	60
00505 009 06348	US101(HWY009) over JUNO OXING SPRR at 020 MI N TILLAMOOK	54
07794A001 28225	I-5 (HWY 001) NB over HWY 51 at 020 MI N MARION-CLACK LN	54
07029A000000110	NE HALSEY ST over B-21 X I-84/UPRR/LT RAIL at NE 68TH AVE & NE HALSEY	53
16161 001 24938	I-5 (HWY 001) NB over COMMERCIAL STREET at 00.33 M JUNCTION HWY 1E	52
07439A001 25257	I-5 (HWY 001) SB over MILL CREEK at AT SALEM SCL	49
09255 061 00139	SW 12TH ST over HWY 61 at 0.2 MI S OF MARKET ST	48
13514C064 02271	I-205 (HWY 064) over HWY 64 CONN 2 at 3.8 MI S ORE-WASH LINE	48
08198 001 29755	SW BRIER PLACE over HWY 1 I-5 at 3.9 MI S OF BURNSIDE BR	47
09724 064 00828	SUNSET AVE over HWY 64 at 0.9 MI S OF WILLAMETTE R	47
02173A035 07602	OR 42 (HWY 035) over CENTRAL ORE RR at 012 MI NW HWY 1 JCT	46
07442 001 25032	BATTLE CREEK RD over I-5 (HWY 001) at 013 MI N OF JCT HWY 1E	46
08333 001 05540	I-5 (HWY 001) over FOOTHILL BLVD at 24.9 MI N OF MEDFORD NCL	46
08676B001C02758	BARNETTE RD CONN over I-5 (HWY 001) at 1.5 MI N OF MEDFORD SCL	46

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