AN ABSTRACT OF THE THESIS OF

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Title: The Influence of Seismic Attack and Chloride-Induced Corrosion on a Life Cycle Inventory Assessment of Different Concrete Mixtures

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The life cycle assessment (LCA) process is a systematic approach to determining the environmental impacts of different products and processes. LCA is a relative approach that requires functional equivalence for the results to be compared. A method is presented that achieves functional equivalence by equating the reliability indexes being compared. An example of the method is completed on a hypothetical bridge located in Astoria, OR. The bridge is modeled for three possible construction materials: concrete with ordinary portland cement, concrete with recycled concrete aggregate and concrete with high volume fly ash. These three materials have different mechanical properties that affect the seismic resilience of the bridge as well as the degradation of the structure over time. The reliability index for a bridge made out of each material is determined by modeling the occurrence of a Cascadia Subduction Zone earthquake, and accounting for the deterioration of the bridge due to chloride ingress. Accurate data on the modeling parameters of recycled concrete aggregate and high volume fly ash are not well documented, so an incremental approach is taken. The recycled concrete aggregate is modeled with increasing coefficients of variation for compressive strength, and the high volume fly ash is modeled with increasing resistance to chloride ingress. Compressive strengths required to achieve the same reliability index as the ordinary portland cement model are calculated using structural reliability methods. Simplified mixture designs are presented for each material and a life cycle inventory assessment is completed. The life cycle inventory assessment data, based on carbon emissions, energy and virgin aggregate usage, are then compared. The objective of this project is to determine the importance of achieving functional equivalence for an LCA, and to present a simplified method for how this can be done. For this reason the life cycle inventory assessment data are compared to a mixture design that does not achieve functional equivalence. The method shows a decrease in the reliability of a recycled concrete aggregate mixture design, which requires an increase in the compressive strength to achieve functional equivalence. An increase in compressive strength produces more carbon emissions, uses more energy and uses less virgin aggregate. The high volume fly ash mixture design requires a decrease in the compressive strength which reduces the amount of carbon emissions and energy use and increases the amount of virgin aggregate required. The fluctuations in the carbon emissions and energy usage show the importance of considering

functional equivalence properly in an LCA. Areas in which improvement can be made are identified and discussed.

The Influence of Seismic Attack and Chloride-Induced Corrosion on a Life Cycle Inventory Assessment of Different Concrete Mixtures

by
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APPROVED:

Major Professor, representing Civil Engineering

Head of the School of Civil and Construction Engineering

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I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Joshua W. Christiansen, Author

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Introduction

Sustainability

> Humanity has the ability to make development sustainable - to ensure that it meets the needs of the present without compromising the ability of the future generations to meet their own needs. The concept of sustainable development does imply limits - not absolute limits but limitations imposed by the present state of technology and social organization on environmental resources and by the ability of the biosphere to absorb the effects of human activity - Brundtland (WCED, 1987)

The above quote is often used to define sustainability. The quote offers a broad definition of sustainability and implies that sustainability is a worldwide issue. Brundtland provides insight on the importance of sustainability: to meet the needs of the present without compromising future generations from meeting their own needs. The standard of living in the United States is very high, but the standard of living is based on consumption. The consumption rates of natural resources in the U.S. are high, the U.S consumed 18.7 million barrels of oil in 2009 (US DOE, 2009), and with a limited amount of oil in the world, a shift from consumption to sustainability is needed to maintain the current standard of living.

As civil engineers, we have a large effect on the world around us. Roads, bridges, dams, buildings, and sewers are all critical to the current standard of living and are designed and maintained by members of our field. Infrastructure also has the negative effect of consuming valuable resources and contributes to the pollution of our environment and to climate change. Mehta (2001) reports that the concrete industry produces 1.6 billion tons of cement a year which accounts for 7% of the global loading of carbon dioxide into the atmosphere. To reduce carbon emissions and waste in the construction industry, the field of civil engineering needs to improve the efficiency in the design of infrastructure.

To improve upon efficiency in design, measurement techniques are needed. Structures have many environmental impacts that can be measured. One such measure is embodied energy. Embodied energy is defined as the non-renewable energy consumed in construction materials and is typically broken into: initial embodied energy and recurring embodied energy. Initial embodied energy is a representation of the non-renewable resources consumed in acquisition, processing, manufacturing and transportation of raw materials as well as construction. Recurring embodied energy is a representation of the non-renewable resources consumed in maintenance, repair and replacement of components and systems of a structure (Kesik, 2002). Recurring embodied energy is not to be confused with operating energy, which is the energy required for operation of a structure (e.g., heating, cooling, lighting and ventilation).

Embodied energy is typically measured in energy use per unit weight of material. Typically the use of non-renewable energy is related to other environmental impacts, such as resource depletion and greenhouse gas emissions. Greenhouse gas emissions are another important measure of a structure's negative impact on the environment. Greenhouse gases trap heat in the atmosphere, which leads to climate change. Although climate change and its consequences are not fully understood, climate change could raise sea levels, create stronger storms and increase floods and droughts (IPCC, 2007). To minimize the impact of climate change, the reduction of greenhouse gases is important.

That embodied energy is consumed during the entire life of a structure, shows the importance of durability to reducing embodied energy. Increased durability leads to reduced maintenance and repair, which decreases energy expenditures, greenhouse gas emissions and the use of non-renewable resources. Durability is often compromised when designers and owners confuse it with the issue of first costs (Kesik, 2002). The initial embodied energy associated with the design phase is considered, while the recurring embodied energy is neglected. If the relationship between durability and embodied energy were better understood, and included in the design phase of a structure, owners would be more likely to evaluate the durability and recurring embodied energy of their structure.

As sustainability moves toward prominence in the construction industry, one system has become the industry standard of sustainability in the United States: the Leadership in Energy and Environmental Design (LEED) rating system. The goals of LEED are to improve building performance in the following areas: energy savings, water efficiency, CO_2 emissions reduction, improved environmental quality and stewardship of resources and sensitivity to the impacts (USGBC, 2009). LEED awards accreditation through using a 100-point scale where credits are given depending on their environmental impact. Similar approaches to the LEED system are being developed by others for infrastructure (ASCE, 2010, Greenroads, 2010).

Complaints have arisen about the LEED system and the question of whether LEED truly measures sustainability has been asked (Scofield, 2009). One of the concerns with LEED is that the measurement scheme is intended to influence the design decisions, without data on the environmental impact of those decisions. Also, LEED has no requirements for the long-term performance of the structure. In considering only the initial phase of a structure's lifespan and disregarding structural maintenance and durability, LEED fails to address important aspects of sustainability. Additionally, without measuring the long-term performance of LEED certified buildings, LEED fails to determine if actual improvements are made. Measurement of data and specific goals are required to determine if improvement in the area of sustainability is actually being achieved.

Measurement of the embodied energy for a structure is a key component of life cycle assessment (LCA). LCA is a technique for determining the environmental impacts of a material throughout the entire lifespan of that material. Compared to LEED, LCA is a more systematic approach to estimating the environmental impact of a structure. LCA addresses the environmental aspects and potential environmental impacts of a structure throughout the entire life cycle, which includes: raw material acquisition, production, use and end-of-life treatment, recycling and final disposal.

Project Framework

To measure the impact a structure has on the environment, the amount of carbon emissions, energy and virgin aggregate usage will be determined. These impacts will be studied in a hypothetical bridge located off the coast of Oregon for three different materials: ordinary portland cement concrete, concrete made with recycled concrete aggregate and high volume fly ash concrete. The addition of fly ash is a common practice that is being promoted as a option to reduce the environmental impact of concrete. Recycled concrete aggregate is not commonly used in structural applications, especially in the United States. Recycled concrete aggregate was used in structural applications in Texas, but due to increased susceptibility to alkali-silica reaction the structures degraded rapidly and recycled concrete aggregate is now banned in Texas. To measure the impacts the life cycle assessment (LCA) process will be used.

LCA requires that the different materials have functional equivalence to be compared. To create different bridge models that are functionally equivalent, the safety of each bridge will be examined and required to be equal. Safety will be measured by the reliability index, which is related to the probability of failure of a structure. Structural reliability methods will be used to obtain the reliability indexes for each bridge model. Completing a life cycle assessment on three bridges with the same reliability indexes will allow for comparison of the environmental impacts of each material.

Organization of Thesis

Chapter 2 of this thesis is a literature review that examines similar works in the field of life cycle assessment and degradation modeling. Mechanical properties of concrete, chloride ingress, alkali-silica reaction and seismic hazards are reviewed. The literature review concludes with topics in life cycle assessment standards. Chapter 3, Methods and Materials, discusses how the bridge is designed and modeled, and the reliability methods used to determine reliability indexes. The framework of the life cycle assessment concludes chapter 3. In chapter 4, the results chapter, the reliability indexes are presented for each material along with the carbon emissions, energy usage and virgin aggregate usage determined from the life cycle inventory assessment. In chapter 5, Discussion and Conclusion, the methods used and the importance of this work are discussed. Improvements that can be made and issues that arose during modeling are also presented.

Literature Review

Similar Works

Russell-Smith and Lepech (2009) apply life cycle assessment techniques to the retrofitting of bridges. Three different retrofitting methods are compared: steel jacketing of columns, fiber reinforced polymer jacketing of columns using epoxy resins and fiber reinforced polymer jacketing using unsaturated polyester resin. A three span bridge structure is modeled with the seismic risk located in Los Angeles County, USA. Fragility curves are used to quantify the impact of structural retrofitting to prevent damage or collapse from a given seismic event. Fragility curves compare an intensity factor (e.g. peak ground acceleration) to the probability of exceedance of a particular damage state. The "cost", carbon emissions and energy use, of each retrofitting scheme is then determined using the following equation:

$$E\left[C(t,X)\right] = C_0 + (C_1P_1 + C_2P_2 + ...C_kP_k)\frac{N}{\lambda}(1-e^{-\lambda t})$$

where E is the expected life cycle cost, C is the life cycle cost of a retrofit strategy, X, over a specified length of time, C_0 is the initial cost, C_k is the k^{th} damage state failure cost in present value and , P_k is the probability of the k^{th} loading state being reached at the time of loading, λ is the annual discount rate, N is the annual event occurrence rate and k is the total number of damage states under consideration. The results for carbon emissions and energy use are presented per year for the expected lifespan of the structure.

This study does not use the method presented by Russell-Smith and Lepech. The method developed by Russell-Smith and Lepech is not conducive to time dependent degradation effects of structures that are considered in this project. Finally, the paper was unclear on exactly how many terms were used in the above equation, and what simplifications were used.

Ghosh and Padgett (2010) offer the formulation of time-dependent seismic fragility modeling. The study considers the aging of typical highway bridges by including probabilistic models of chloride induced corrosion of reinforced concrete columns and steel bridge bearings. The degradation model for corrosion is based off of the model presented by Stewart and Rosowsky (1998), which will be described later in the literature review. Modeling a multi-span continuous steel girder bridge, Ghosh and Padgett determine that there is a significant increase in the bridge system vulnerability over time due to aging. Their findings highlight the importance of considering the effects of aging and deterioration on the seismic vulnerability of bridges. Ghosh and Padgett suggest that such models offer more realistic estimates of corroded bridge seismic vulnerability and enable more accurate estimates of potential damage, life cycle cost, and needed rehabilitation.

This project attempts to combine the time-dependent modeling described by Ghosh and Padgett, with a life cycle assessment based on specific hazards similar to that by Russell-Smith and Lepech. The life cycle assessment method based on specific hazards is a new approach to life cycle assessments with few publications to review.

Probabilistic Mechanical Properties of Concrete

Ordinary Portland Cement

Mirza et al. (1979) state that due to variations in material properties, proportions of the concrete mixture, curing methods and testing procedures, it is expected that the strength of concrete will differ from the design strength. After reviewing available test data, Mirza et al. conclude that the coefficient of variation (COV), which is defined as the ratio of standard deviation to the mean, can be taken as roughly constant at 10%, 15% and 20% for strength levels below 4000 psi for excellent, average, and poor control. For concrete strengths above 4000 psi, the standard deviation remains approximately constant at 400 psi, 600 psi and 800 psi for excellent, average and poor control. Definitions of excellent, average and poor control are not provided in the article.

Due to different curing and placing procedures, effects of vertical mitigation of water, different sizes and shapes, and different stress patterns, the concrete in-situ strength tends to be lower than the strength of test cylinders. This can be offset by the fact that cylinder strength is often 700 to 900 psi greater than the design strength of the structure (Mirza et al. 2009). Mirza et al. present an equation to determine the coefficient of variation for in-situ compressive strength ($COV_{in\text{-}situ}$) from the coefficient of variation for cylinder compressive strength (COV_{cyl}):

$$COV_{in-situ} = (COV_{cyl}^{\ 2} + 0.0084)^{0.5}$$

Low and Hao (2001) investigate the reliability of reinforced concrete slabs under blast loading. The assumption of good control is used in this paper, as well as the assumption that the COV of cube strength and cylinder strength are the same. The COV of cube strength was taken to be 0.07. The above equation is referenced to determine the in-situ coefficient of variation for compressive strength and a value of 0.11 is used, with a normal distribution and a mean value of 51.2 MPa (7400 psi).

Frangopol et al. (1997) present a reliability-based approach to the design of reinforced concrete bridge girders that are under corrosion attack. Frangopol et al. use a mean compressive strength of concrete of 27.6 MPa (4000 psi) and a COV of 0.15. No distribution is given.

Val & Chernin (2009) studied the effects of corrosion on deflections of reinforced concrete beams and the probability of serviceability failure due to excessive deflection. The concrete compressive strength is modeled with a mean value of 38 MPa (5500 psi), a COV of 0.16 and a lognormal distribution.

Marsh and Frangopol (2008) use structural health monitoring techniques to determine the corrosion rate. For the modeling process Marsh and Frangopol use a compressive strength of 19.03 MPa (2800 psi) with a standard deviation of 3.43 MPa (500 psi) which corresponds to a COV of 0.18.

Recycled Concrete Aggregate

Xiao et al. (2005) present a statistical analysis of the compressive strength of recycled concrete aggregate (RCA), with aggregate coming from a single source. The statistical characteristics of RCA are still not well understood, partly due to the range of the quality and composition of demolished concrete. Often the assumption is made that the COV of the compressive strength of RCA is larger than that of normal concrete. Xiao et al. point out that past lab tests have not been conclusive in determining values of the COV of RCA. Some lab tests resulted in similar COV for RCA and normal concrete, while other tests resulted in significantly larger COV, in some cases even up to 20% larger compared to normal concrete. The test results from Xiao et al. are given based on the percent of recycled coarse aggregate in the mixture design. For no recycled coarse aggregate, the mean compressive strength is 41.6MPa (6000 psi) and the COV is 0.083, while 30% recycled coarse aggregate had a mean compressive strength of 41.5MPa (6000 psi) and a COV of 0.095. For a mixture design with 50% recycled coarse aggregate the mean compressive strength was 40.2 MPa (5800 psi) and the COV was 0.097, while 100% recycled course aggregate had a mean compressive strength of 36.5 MPa (5300 psi) and a COV of 0.082. Xiao et al. conclude that the compressive strengths are "not much different from those of the normal concrete," though the 100% replacement rate does result in a loss of compressive strength. In addition, Xiao et al. claim that both the normal and lognormal distribution models can be applied to fit the compressive strength distribution.

Mukherjee et al. (2003) study twenty recycled aggregate cubes and compared them to normal concrete. The conclusion of this test is that the RCA shows inferior mean strength compared to the normal concrete and that the statistical distributions in general do not differ very much from that of normal concrete.

In both cases, these statistical parameters are for specific mixture designs created under laboratory conditions. Testing concrete strength under lab conditions will have less variation in the strength due to the amount of control. Testing a specified concrete in-situ strength will have less control and more variation in the mixture design, mixing and pouring conditions. For these reasons the COV for lab tested cylinders is assumed to be less than the COV for a specified concrete in-situ strength.

High Volume Fly Ash Concrete

Aggarwal et al. (2010) complete a literature review of statistical properties of high volume fly ash for India. Aggarwal et al. note that the properties of fly ash vary from different sources and can even vary from a particular source over a period of time. In one test, 40% of the cement was replaced by fly ash and achieved an increase in strength of concrete of 23% and 38% at 28 days and 56 days. In another test, 40% of the cement was replaced by fly ash and achieved 45 MPa (6500 psi) characteristic strength at 28 days. The overall trend of partial replacement of cement by fly ash in concrete results in a decrease in compressive strength, split tensile strength, modulus of elasticity and abrasion resistance at 28 days of age. However, all these properties of hardened concrete showed significant improvement at 90 days and thereafter when compared to concrete with no fly ash.

Poon et al. (2000) also discuss that fly ashes generally have negative effects on the concrete strength, particularly at early ages. However fly ash concrete may have better strength performance when prepared at lower water to binder ratios, where the binder content includes cement and fly ash. Tests on concrete with a water to binder ratio equal to 0.5 and a 45% class F fly ash replacement resulted in about 30% reduction in 28-day compressive strength. Concrete with a water to binder ratio equal to 0.3 resulted in strength reduction around 17%.

High volumes of fly ash in concrete mixture designs affect the diffusion of chloride through the material. Sujjavanich et al. (2005) test chloride permeability of high volume fly ash (HVFA) concrete mixture designs and conclude that HVFA concrete has lower chloride permeability and has a tendency to minimize corrosion risk.

Patel et al. (2004) also note the rapid chloride permeability appears to be low to very low as defined by ASTM C 1202-97 and ranged from 772 to 1379 coulombs. Low permeability was achieved with fly ash replacement rates from 30% to 60%. These tests illustrate that fly ash can improve the permeability of concrete due to its capability of transforming large pores of concrete into small pores and reducing micro-cracking in the interfacial transition zone.

No literature was found discussing typical probabilistic distributions for the compressive strength of high volume fly ash. The distribution will be assumed to be similar to that of a ordinary portland cement mixture design.

Concrete Degradation Model

Chloride

The chloride diffusion process is divided into two different phases. In the initiation phase, chloride diffuses through the concrete but has yet to cause any damage to the reinforcing steel. Once the amount of chloride exceeds the critical chloride concentration required to dissolve the protective passive film around the reinforcing, deterioration of the steel begins. The propagation phase begins once the critical chloride concentration is met. Stewart & Rosowsky (1998) determine the time to initiation by using:

$$C(x,t) = C_0\left[1-\operatorname{erf}\left(\frac{x}{2\sqrt{tD}}\right)\right]$$

where *C(x,t)* is the chloride content at a distance *x* from the concrete's surface at time *t*, C_0 is the surface chloride content, *D* is the apparent diffusion coefficient and erf is the error function. The values used in the above equation include 2.0 x 10^{-8} cm^2/s (3.1 x 10^{-9} in^2/s) for the mean diffusion coefficient with a 0.75 COV and a lognormal distribution, 3.5 kg/m^3 (0.218 lb/ft^3) for the mean surface chloride concentration with a 0.5 COV and a lognormal distribution. The critical chloride concentration required to initiate corrosion of reinforcement is uniformly distributed between 0.6 kg/m^3 and 1.2 kg/m^3 (0.037 lb/ft^3 and 0.075lb/ft^3).

Once the critical chloride threshold is reached, the deterioration of reinforcing steel begins. The model used by Stewart and Rosowsky assumes a linear deterioration of the reinforcing given by the following equation:

$$d(t) = \begin{cases} d_i & \text{if } \ t \le T_i \\ d_i - 2\lambda(t-T_i) & \text{if } T_i < t \le T_i + (d_i / 2\lambda) \\ 0 & \text{if } \ t > T_i + (d_i / 2\lambda) \end{cases}$$

where d_i is the initial bar diameter, T_i is the time to initiation and λ is the corrosion rate. The corrosion rate is uniformly distributed between 0.116 mm/year and 0.232 mm/year (0.0005 in/year and 0.0009 in/year).

Alkali Silica Reactions

The alkali silica reaction (ASR) can differ depending on the mixture design. The possibility of using recycled concrete aggregate that has previously been effect by ASR will increase the susceptibility of the concrete to ASR. For a mixture design with fly ash, the susceptibility to ASR decreases, Ichikawa (2009) states that the use of very fine alkali-reactive siliceous admixtures such as fly ash can suppress the alkali silica reaction.

Bažant & Steffens (2000) attempt to model the kinetics of the alkali silica reaction in concrete. ASR creates a gel that expands and causes cracking of the concrete. The problem is complex, influenced by many factors, including the kinetics of the chemical and diffusion processes involved, and the mechanical damage to the concrete, which calls for fracture mechanics. Presently, there are no mathematical models for describing the structural effects of ASR, so ASR was not included in the degradation models for concrete. Current doctoral studies currently at Ècole Polytechnique Fèdèrale de Lausanne are developing models for ASR in concrete.

Seismic Hazards

Cascadia Subduction Zone

Although there have been no recorded large magnitude earthquakes from the Cascadia Subduction Zone (CSZ), there is strong evidence that points to large magnitude earthquakes occurring in this region within the last 400 years. Interpretation of geological evidence, subsidence and tsunami deposits, along the coast suggests that large magnitude earthquakes have occurred in the CSZ (Atwater, 1996). Turbidites, which are deposits on the ocean floor formed by massive slope failures, were cored off the Washington-Oregon coast and revealed as many as 16 slope failures that may have resulted from great earthquakes in the CSZ between 7500 and 300 years ago (Adams, 1990). The turbidites show these sixteen slope failures occurring after the eruption of Mount Mazama, the eruption that formed crater lake in Oregon, which occurred nearly 6600 years ago. These sixteen events correspond to an average return period of 410 years (Adams, 1990).

Marsh & Gianotti (1994) studied the structural response from a large scale CSZ earthquake. To determine the structural response an acceleration-time history is required, but since no subduction earthquakes have occurred in the Pacific Northwest during recorded history, no such records exist. In addition, there are no near source acceleration records for the largest subduction earthquakes in recent times: the 1960

Chile and the 1964 Alaska earthquakes. To analyze structural response, Marsh and Gianotti made artificial ground motions to substitute for actual acceleration records. Marsh and Gianotti considered duration as a key parameter in creating artificial acceleration records. Large magnitude earthquakes have long durations of strong shaking that impact structural response. The Alaskan earthquake (M9.2) was estimated to have strong shaking around two minutes and an overall duration of four minutes. The 1960 earthquake in southern Chile lasted for approximately three and a half minutes.

Atkinson and Macias (2009) modified data from 300 strong motion stations that recorded the Tokachi-Oki main shock, a magnitude 8.2 earthquake that occurred 80 km offshore of Japan in 2003, to produce artificial time histories for a Cascadia Subduction Zone earthquake. The strong motion stations ranged from 40 km to 500 km from the epicenter of the earthquake. The use of a large number of records as a baseline distinguishes this study from other ground motion predictions for the CSZ. Two different methods for creating time histories were used: stochastic methods and modification methods. The stochastic methods are a simplistic way to simulate time histories and may be missing potentially important coherent pulses and phasing information found in real records (Atkinson & Macias, 2009). To include this information, Atkinson and Macias modify the Tokachi Oki time histories to time histories with similar spectral content expected of the CSZ. The modified Tokachi-Oki time history is used in this project's modeling process to represent a possible Cascadia Subduction Zone earthquake. The authors have made the acceleration records readily available in order to compare the different structural response between the stochastic models and the modified records.

Oregon Department of Transportation assessed the seismic vulnerability of the state's highway bridges (Albert et al., 2009). The report estimates the peak ground acceleration possible due to a full length Cascadia Subduction Zone earthquake. The report determines that the region of Astoria, OR can expect a peak ground acceleration between 0.36g - 0.40g. The modified Tokachi-Oki acceleration record used in this project has a peak ground acceleration of 0.36g.

Life Cycle Inventory Assessment

Introduction

Life Cycle Assessment (LCA) is a technique for determining the environmental impacts of manufactured and consumed products. ISO 14040 (2006) defines the purposes of an LCA as: identifying opportunities to improve a product's environmental performance, assisting decision-makers, and helping market eco-friendly materials. There are four phases to a LCA: the goal and scope definition phase, the inventory analysis phase, the impact assessment phase, and the interpretation phase. The goal and scope phase requires the definition of the intended application, the reason for the study, the intended audience and the intent of the results. The inventory analysis phase involves the data collection and calculation procedures to quantify relevant inputs and outputs. The impact assessment phase involves evaluating the significance of the results from the inventory analysis phase. The interpretation phase involves providing results that are consistent with the goal and scope phase and which reach conclusions, explain limitations and provide recommendations. If the impact assessment phase is not completed, the study is instead called a Life Cycle Inventory Assessment (LCI). For this project an LCI will be conducted.

Section 4.1 of IS0 14040 addresses principles of an LCI. One principle is the relative approach and functional unit:

> LCA is a relative approach, which is structured around a functional unit. This functional unit defines what is being studied. All subsequent analyses are then relative to that functional unit, as all inputs and outputs in the life cycle inventory analysis and consequently the life cycle impact assessment profile are related to the functional unit.

The functional unit is defined as a quantified performance of a product system for use as a reference unit. The primary purpose of a functional unit is to provide a reference for which inputs and outputs are related. A functional unit is necessary to ensure comparability of LCI assessment results. For this project the functional unit is the reliability index of a three span bridge. The reliability index provides a quantifiable measure of the safety and reliability of the bridge. Since the LCI assessment is a relative approach, the reliability indexes will also be relative, required to be equal to the reliability index of the bridge constructed with ordinary portland cement. The reliability indexes determined in this project are nominal values and are not representative of the actual probability of failure of the bridge.

Materials and Methods

Bridge Design

The hypothetical bridge is located in Astoria, Oregon near the coast but not in direct contact with saltwater. This location provides for the possibility of chloride ingress due to coastal exposure and the possibility of a CSZ earthquake occurring during the 75 year lifespan of the bridge. The bride is a three span, continuous, cast-in-place, reinforced concrete, tee girder bridge. The spans for this bridge are 8.5 feet, 34 feet and 8.5 feet. There are two substructure supports that stand approximately 16 feet tall. An elevation view of this structure is shown Figure 1.

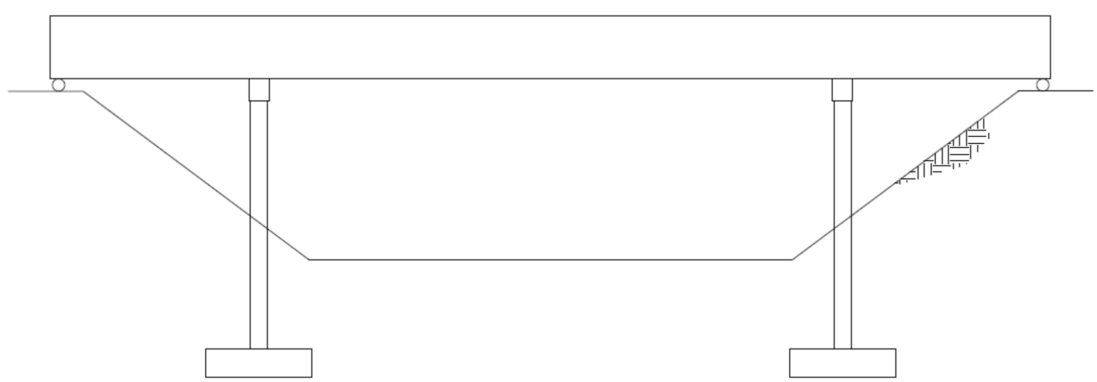

Figure 1 - Bridge Elevation

The bridge is designed using the American Association of State Highway Transportation Officials (AASHTO) load and resistance factor design (LRFD) bridge design specifications, 4th edition (AASHTO, 2007). The design steel yielding stress is 60 ksi and the design concrete compressive strength is 4000 psi. The dead load of the components is 1.44 kip/foot per girder, and the dead load of the wearing surface is 0.33 kip/ft per girder.

The sections of the bridge include a girder section and a column section. The girder is designed for a positive moment region with two rows of three #11 bars and a negative moment region with two rows of three #11 bars. For simplicity, these two unique sections were modeled as one section shown in Figure 2. The column section is a twenty inch square column, with reinforcing consisting of eight #8 bars.

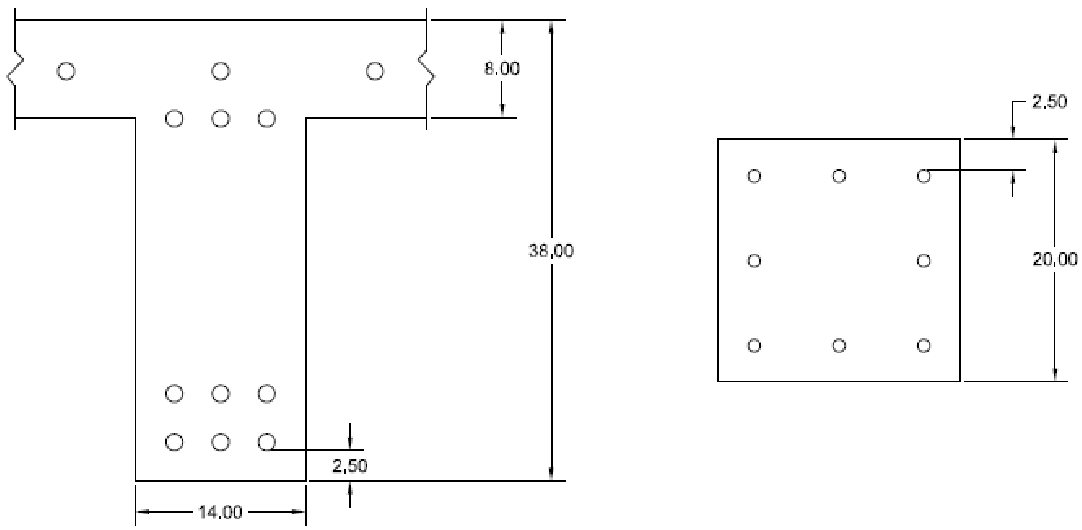

Figure 2 - Bridge Sections

The performance state will separate the safe response of the bridge from the failure response of the bridge. The performance limit for the bridge is set to 0.004 strain, which represents the onset of crushing in the concrete. This value will coincide with significant drift in the deck, and significant damage to the structure.

Bridge Modeling

The bridge is modeled using OpenSees (McKenna et al., 2000), the Open System for Earthquake Engineering Simulation. The bridge is modeled with fixed connections at the base of the columns and roller connections located at the edge of the deck. The mass of the bridge includes the dead load of the components and wearing surface plus an additional 50% to account for live load. The concrete material is modeled with the material command Concrete01 which models a uniaxial Kent-Scott-Park model with degraded linear unloading and loading stiffness and no tensile strength (McKenna et al., 2000). Concrete confinement is modeled in the column core and unconfined concrete is modeled at the edges of the column. The girder concrete is modeled as unconfined concrete. The steel material is modeled with a linear representation for the elastic tangent and a linear increasing representation of strain hardening using the Steel01 command. The girder and column sections are represented with a fiber model. This model generates fibers with a specific material, area and location. The girder and column elements that span from node points are modeled with displacement elements. This element is based on the displacement formulation and considers the spread of plasticity along the element.

The acceleration time history input into OpenSees is the modified Tokachi-Oki earthquake, as discussed in the previous chapter. This time history is applied to the structure only in the longitudinal direction. Stress and strain at the edge of the confined concrete is recorded for use in the reliability analysis.

Structural Reliability Background

The reliability index is an essential component of limit-state design, where the limit state divides the structural response into safe and failure states. Limit state design is represented by two unique probabilistic distributions, the probability that a certain load will be felt by the structure and the probability of the materials capability to resist load. The loads used to determine this distribution include typical truck and vehicle loads, wind loads, ice loads, pedestrian loads and seismic loads. The resistance distribution is often comprised of compressive strength and elastic modulus of concrete, the yield strength and elastic modulus of steel, the actual size of specific members and so on. The resistance distribution (*R*) can be subtracted from the load distribution (*S*) creating a new distribution, the limit state function (*Z*). Failure occurs when the load is greater than the resistance, or when Z is less than zero. In this case the failure surface, which separates the safe domain and the failure domain, is Z=0.

$$p_f = P(Z \leq 0)$$

Using the standard normal distribution function $\Phi(z)$ which has a mean of zero and a variance of one, the probability of failure becomes:

$$p_f = \Phi\left(\frac{0-\mu_z}{\sigma_z}\right) = \Phi(-\beta)$$

where μ_z is the mean of the *Z* distribution, σ_z is the distribution, β is equal to μ_z/σ_z and is referred to as the reliability index (Melchers, 1999).

To determine the reliability index in this project, both the first order reliability method (FORM) and sampling methods are used. FORM determines the reliability index by transforming the distribution into standard normal space and linearizing the limit state function. This linearization occurs at the design point, which is the closest point of the failure surface to the origin. The reliability index can be interpreted as the distance of the design point from the origin.

The FORM analysis has several limitations. The method assumes that the response of the limit state function is not highly nonlinear. Also the method assumes that the slope of the limit state function is continuous, which does not allow for max and if statements in any equations (Haukaas, 2011). Due to the limitations of FORM, sampling methods are also used in this project.

Sampling uses random number generation to determine probabilistic distribution for the response of the structure, which is used to determine the probability of failure. The accuracy of the sampling procedure is determined by the coefficient of variation of the probability of failure. The target coefficient of variation of the probability of failure for this project is set to 0.05, which is usually considered acceptable (Haukaas, 2011). Sampling of this type is also referred to as Monte Carlo Simulation.

Reliability Modeling

Reliability Tools (Mahsuli, 2006) was developed at University of British Columbia and is designed to interface with OpenSees. Reliability Tools (Rt) defines probabilistic distributions for variables that are input into OpenSees. Rt then uses the user-defined output from OpenSees to determine the reliability index. Two methods are used to determine the reliability index: FORM (with gradient determined by finite differences) and sampling.

In Rt, two different models were created. One model is designed to compare the bridge constructed of ordinary portland cement (OPC) to the same structure constructed of recycled concrete aggregate (RCA). The other model is designed to compare the bridge constructed of OPC to the same structure constructed of high volume fly ash (HVFA) cement.

To compare OPC to RCA, the parameters defined in Rt are the mean of in-situ compressive strength and the coefficient of variation. The chloride diffusion parameters are assumed to be the same for OPC and RCA the corrosion was not modeled. The design compressive strength for the bridge model is 4000 psi, per the bridge design. The design compressive strength is the minimum compressive strength required and is lower than the mean in-situ compressive strength. A value of 4500 psi is used to represent the mean value of the concrete in-situ strength. The concrete strength in OPC was defined with a lognormal distribution and coefficient of variation of 0.1. The model assumes concrete strength does not vary with location. Since a specific value for the coefficient of variation in RCA is unknown, the coefficient of variation used to model RCA is increased to 0.12, 0.14 and 0.16 to represent possible COV values. The mean compressive strength for the RCA will then be determined, based on matching the reliability index of the RCA to the reliability index of the OPC model. To minimize computational time FORM is used to determine the reliability index for the OPC and RCA models.

To compare OPC to HVFA an additional parameter, decrease in bar diameter, is added into Reliability Tools. The decrease in bar diameter is determined by the chloride diffusion model prescribed by Stewart and Rosowsky. A Monte Carlo simulation is used to combine multiple probabilistic distributions into one distribution. The previous probabilistic distributions mentioned in the literature review: surface chloride content, apparent diffusion coefficient, critical chloride concentration and corrosion rate are part of the sampling. The values used for the sampling are shown in Table 1. Two additional parameters are added to the Monte Carlo simulation, concrete cover and probability of earthquake occurrence. The distribution for concrete cover has a mean value of 5.08 cm (2 inches) and a COV of 0.1. Time of occurrence of Cascadia Subduction Zone earthquake is represented by a Weibull distribution as suggested by Rikitake (1999). The Weibull distribution has a scale parameter of 500 and a shape parameter of 5 as defined by the program MATLAB. Because an earthquake is assumed to occur during the lifespan of the bridge, the resulting distribution is truncated at values outside the service life and the resulting distribution is then scaled upward.

Table 1 - Properties of Random Variables

Variable	Distribution	Mean	COV
Surface Chloride Content	lognormal	2.95 kg/m^3	0.5
Apparent diffusion Coefficient	lognormal	2.0e-8 cm^2/s	0.75
Critical Chloride Concentration	Uniform (0.6 -1.2)	0.9 kg/m^3	-
Corrosion Rate	Uniform (0.116 -0.232)	0.174 mm/year	-
Concrete Cover	lognormal	5.08 cm	0.1
Time of earthquake	Truncated Weibull	See Text	

Combining these inputs provides an output for the probabilistic distribution for the reduction in bar diameter. Figure 3, represents the output of the Monte Carlo simulation with one million samples. The sampled distribution is represented by the thin line while the thick line represents the distribution input into Reliability Tools. The distribution is a lognormal distribution with a mean of 3.14 and a COV of 0.53. The negative of the distribution is shifted 3.03 to make the plot seen in Figure 3. The equation below is used to determine the distribution.

$$\text{Fit} = \text{Shift} - \text{lognormal}(\text{mean}, \text{COV})$$

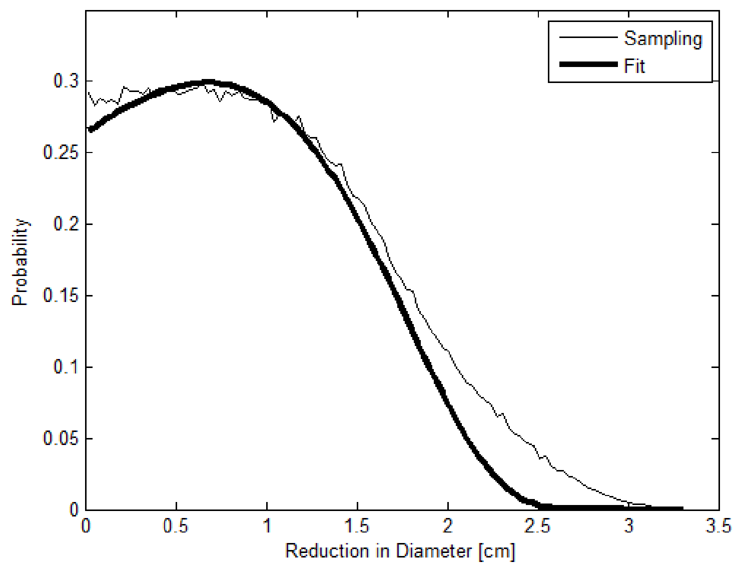

Figure 3 - Reduction in bar diameter due to corrosion

The diffusion of chloride is reduced by the addition of fly ash into the cement mixture. Since no specific values are available in the literature for the mean diffusion coefficient, creating specific bar diameter distributions are not possible (Life - 365 (American Concrete Institute, 2008) is a resource that was not available during this project but could provide information on diffusion coefficients for fly ash mixture designs). To model the bridge constructed with high volume fly ash the mean value for the apparent diffusion coefficient is reduced by factors of 0.8, 0.6 and 0.4 to represent a range of possible distributions. The coefficient of variation of the diffusion coefficient remains constant. Each distribution for bar diameter is shown in Figure 4, 5 and 6 with the fitted lognormal distribution. The 0.8 factor fitted distribution has a shift of 2.98, a mean of 3.66 and a COV of 0.54. The 0.6 factor fitted distribution has a shift of 3.03, a mean of 3.70 and a COV of 0.53. The 0.4 factor fitted distribution has a shift of 3.20, a mean of 5.05 and a COV of 0.55.

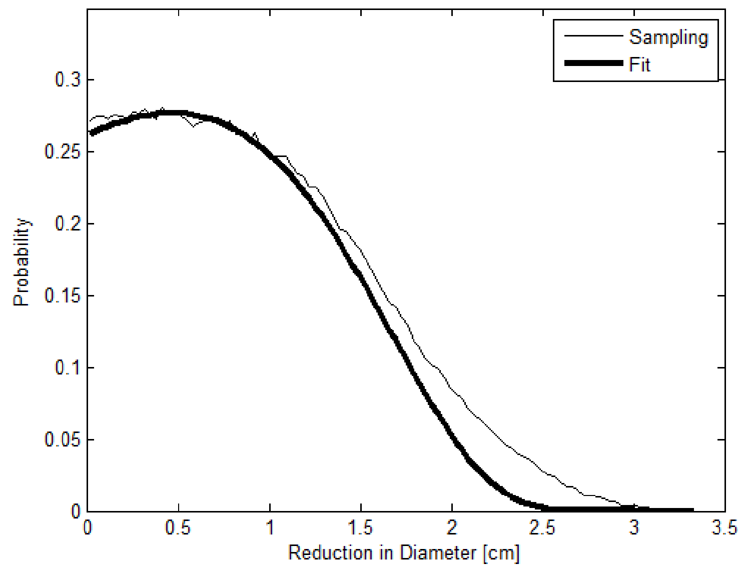

Figure 4 - Reduction in bar diameter due to corrosion (0.8 diffusion coefficient)

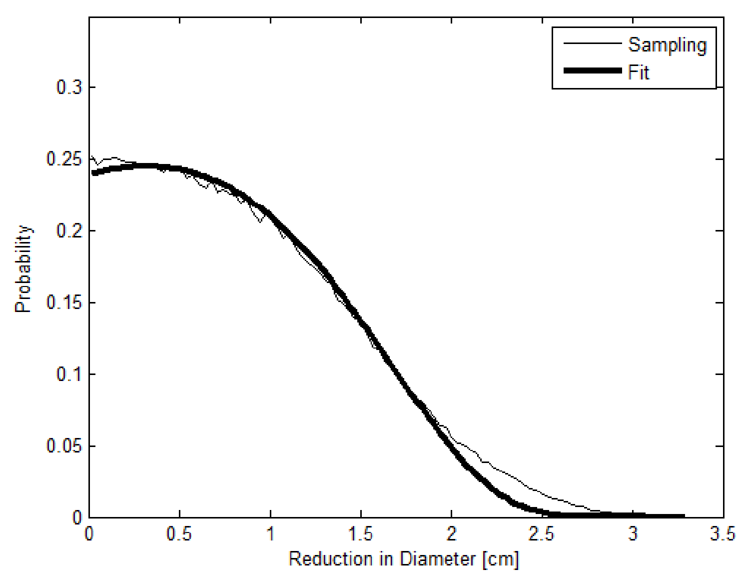

Figure 5 - Reduction in bar diameter due to corrosion (0.6 diffusion coefficient)

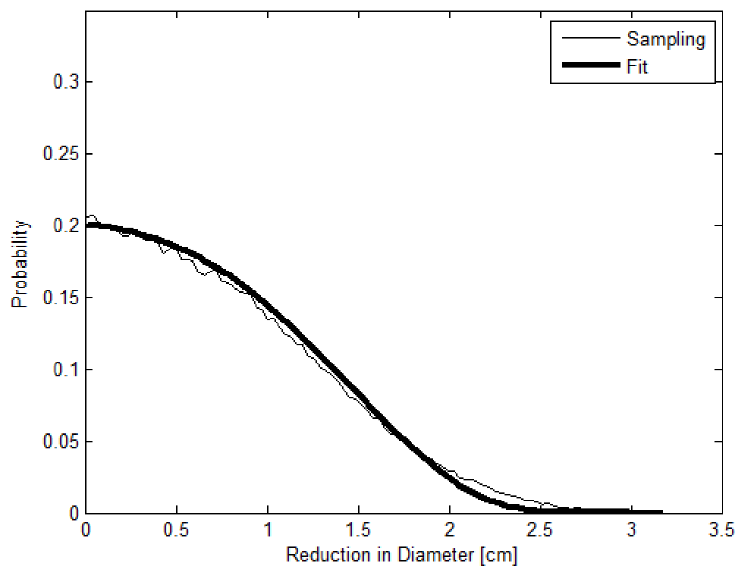

Figure 6 - Reduction in bar diameter due to corrosion (0.4 diffusion coefficient)

Adding the additional parameter of reduction in bar diameter made using a FORM analysis inadequate. FORM is based on the assumption that the limit state is not highly nonlinear. Multiple input variables and the dynamics of the structure create a highly nonlinear response for which FORM would not converge. To determine the reliability index for the high volume fly ash concrete a sampling procedure was run in Rt.

Life Cycle Inventory Assessment

Resources

The life cycle inventory assessment is a complex procedure that requires information from a variety of sources. The BATH - ICE V2.0 Database (Hammond & Jones, 2009) is used to determine the energy use for virgin aggregate manufacturing, and recycled aggregate manufacturing and the CO_2 emissions for virgin aggregate manufacturing. The National Renewable Energy Laboratory (US Department of Energy, 2008) is used to determine the energy use for cement production and the CO_2 emissions for cement production and transportation. Marceau et al. (2006) provide values for the energy use for transportation while Marceau et al. (2007) provide values the energy use for concrete manufacturing. Shima et al. (2005) provide values for the CO_2 emissions for recycled aggregate manufacturing. Athena Sustainable Materials Institute (2005) was used to determine the CO_2 emissions for concrete manufacturing. The above sources are referenced because they present the data clearly and are respected sources. A source that was reviewed but not used in the project includes Worrell et al. (2001).

To accurately determine the transportation requirements for a bridge construction located in Astoria, OR, the closest applicable production sites were used. Fly ash production is located in Centralia, WA, HVFA concrete production is located in Seattle, WA and RCA manufacturing is located in Colton, CA. Virgin aggregate manufacturing, ordinary portland cement production and concrete manufacturing are assumed to be located in Portland, OR. In actuality, no concrete manufacturing plant is located and in Portland, OR and this location is used in error. The closest concrete manufacturing site is located in Seattle, WA.

Framework

The goal of the life cycle inventory assessment is to evaluate the global warming potential, the energy use and the virgin material use of the three different types of concrete mixtures. To compare the impact of the materials used, the reliability indexes of each bridge structure will be same. Then, based on the materials needed to achieve the same reliability, a life cycle inventory study (LCI study) will be performed for each type of concrete. With similar reliability indexes of the structures, functional equivalence is achieved, allowing for LCI study data to be compared. The LCI study will determine the relative impact on global warming and energy use of each type of concrete. To assess the global warming potential, greenhouse gas emissions will be reported in terms of kg of CO_2 equivalents per kg of concrete. CO_2 equivalent is a measure to compare various greenhouse gasses based on their global warming potential, using CO_2 as the reference gas. Different gasses (e.g. carbon dioxide, nitrous oxide, methane) absorb different amounts of infrared radiation and decay at different rates, meaning they have different warming potential. CO_2 equivalent allow for the warming potential of these different gasses to be reduced into one term, equivalent mass of CO_2 (IPCC, 2007). Energy use will be reported in MJ used per kg of concrete, and virgin aggregate will be measured in pounds per kg of concrete.

This study will look at the product system of a three span bridge structure, from resource extraction, manufacturing of materials, mixing, transportation and use. The construction phase of the structure, maintenance and the end-of-life disposal phase will be the same for each type of concrete and therefore will not be assessed in this study. Areas where the type of concrete differ include the acquisition of raw materials, raw material transportation, manufacturing of materials other that portland cement. For recycled concrete aggregate, the study will begin at the crushing stage, after the recycled concrete has been obtained. The previous stages in the life cycle of recycled concrete aggregate are associated with the deconstruction stages of the previous use of the concrete. The study will include the transportation of the recycled aggregate and the acquisition and transportation of supplemental virgin aggregate, if any. For high volume fly ash concrete, the study will assess the acquisition of the fly ash, the transportation of the fly ash, and the acquisition and transportation of virgin aggregate. For ordinary portland cement concrete the study will include acquisition and transportation of virgin aggregate.

This study is specific to new construction in Astoria, OR, due to the proximity of the Cascadia Subduction Zone and the transportation distances required. All materials are assumed to be readily available.

Results

Modeling

Recycled Concrete Aggregate Comparison

In comparing ordinary portland cement to recycled concrete aggregate the coefficient of variation of the compressive strength of concrete was varied. The initial run of ordinary portland cement has a mean value of 4500 psi and a coefficient of variation of 0.1. Using this model for ordinary portland cement produced a reliability index of 6.24. Recycled concrete aggregate was then modeled with coefficients of variations of 0.12, 0.14, and 0.16. FORM is used to determine the mean compressive strengths required to achieve a reliability index of about 6.24 are then 5100 psi, 5800 psi and 6550 psi, respectively. The results are shown in Table 2. Implications of the large reliability index (β) are discussed in the following chapter.

Table 2 - Compressive strength required for varying COV and maintaining β

Model	f`c (psi)	COV	β
OPC	4500	0.10	6.24
RCA	5100	0.12	6.24
RCA	5800	0.14	6.26
RCA	6550	0.16	6.23

High Volume Fly Ash Comparison

Comparing ordinary portland cement to high volume fly ash concrete added the additional element of the reduction in bar area distribution. Modeling ordinary portland cement with a factor of 1.0 applied to the diffusion coefficient, a mean compressive strength of 4500 psi and a coefficient of variation of 0.10, resulted in a probability of failure of 0.127 and a corresponding reliability index of 1.138. The sampling results for the model with a diffusion coefficient of 0.8 are shown in Figure 7. The results were obtained by varying the mean compressive strength while maintaining the coefficient of variation constant at 0.10.

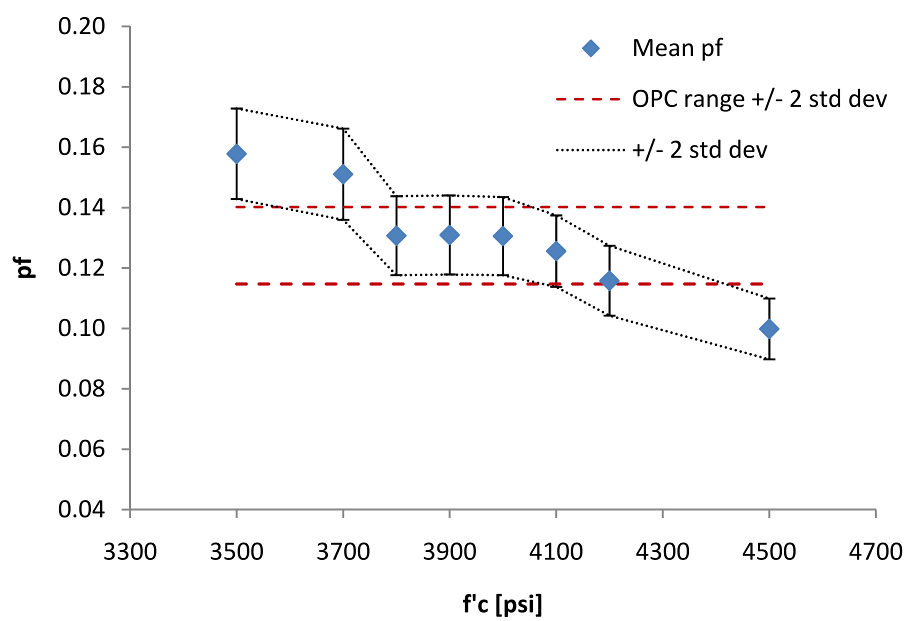

Figure 7 - Sampling results for diffusion coefficient factor of 0.8

For the 0.8 reduction factor, a mean compressive strength of 4100 psi resulted in a reliability index of 1.147 (pf = 0.126), this value was used to compare to the ordinary portland cement model.

The results for the probability of failure for reduction factors of 0.6 and 0.4 are shown in Figure 8 and Figure 9. For the 0.6 reduction factor, a mean compressive strength of 3900 psi resulted in a reliability index of 1.146 (pf = 0.126). For the 0.4 mean diffusion coefficient reduction factor, a mean compressive strength of 3050 psi resulted in a reliability index of 1.150 (pf = 0.125). A summary of the required compressive strengths are shown in Table 3 along with the ordinary portland cement model.

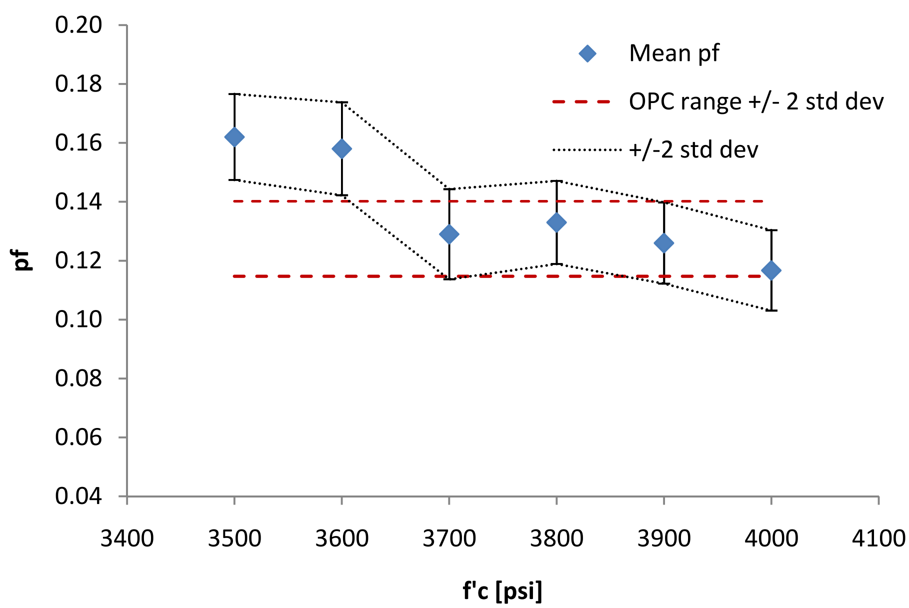

Figure 8 - Sampling results for diffusion coefficient factor of 0.6

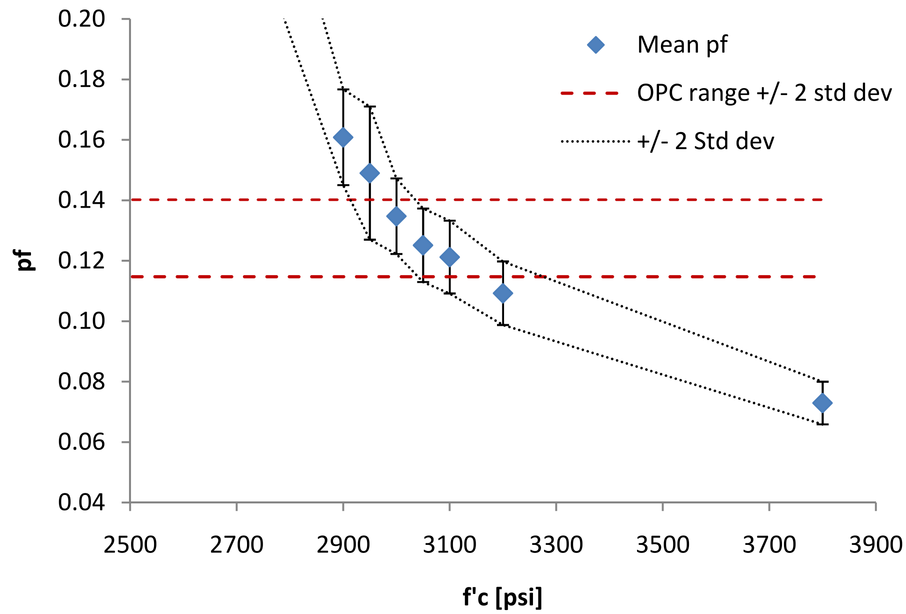

Figure 9 - Sampling results for diffusion coefficient factor of 0.4

Table 3 - Compressive strength required for varying diffusion factor and maintaining β

Model	Diffusion Factor	f`c	β
OPC	1	4500	1.138
HVFA	0.8	4100	1.147
HVFA	0.6	3900	1.146
HFVA	0.4	3050	1.150

Mixture Designs

Ordinary Portland Cement

The mixture designs were developed using ConcreteWorks software Version 2.1.3 (Concrete Durability Center, 2006). The mixture designs call for a 2 inch slump and for air entrainment of 5%. All units for mixture designs are lb/yd^3. The specified compressive strength for the mixture design is 4000 psi which corresponds to the minimum compressive strength. ConcreteWorks is used to determine the mean compressive strength for a design strength of 4000 psi. The inputs are: standard deviation and number of samples taken. Values of 400 psi and 30 samples result in a mean compressive strength of 4500 psi. The mixture design for OPC is shown in Table 4.

Table 4 - OPC Mixture Design

Mean f`c (psi)	Specified f`c (psi)	Water (lb/yd^3)	Cement (lb/yd^3)	Coarse Agg (lb/yd^3)	Fines (lb/yd^3)
4500	4000	308.4	677	1863	991.7

Recycled Concrete Aggregate

All recycled concrete aggregate mixture designs were assumed to be the same as ordinary portland cement mixture designs. The addition of superplasticizers is normally required to create the desired workability but is not considered in the presented mixture designs. The design strengths for each mixture design are 4600 psi, 5300 psi and 6000 psi. For comparison purposes, a baseline case will be presented with the same mixture design as OPC with replacement rates of 50% and 100% of RCA. Mixture designs for RCA are shown in Table 5.

Table 5 - RCA Mixture Designs

Mean f`c (psi)	Specified f`c (psi)	Water (lb/yd^3)	Cement (lb/yd^3)	Coarse Agg (lb/yd^3)	Fines (lb/yd^3)
5100	4600	308.4	771	1863	912.6
5800	5300	308.4	852	1863	844.5
6550	6000	308.4	965	1863	749.4

High Volume Fly Ash

All high volume fly ash concrete mixture designs are for Class F fly ash. The specified compressive strengths are 3600 psi and 3400 psi and 2550 psi, which are lower than the bridge design strength but achieve functional equivalence. For each mixture design a 40% and 80% replacement rate of fly ash is represented. High volume fly ash is defined by replacement rates equal to or greater than 50%, but the replacement rates are chosen to represent the low and high range of replacement. A baseline mixture design for the mean compressive strength of 4500 psi is also provided in Table 6 for comparison. The baseline mixture design represents a mixture design that does not have functional equivalence. For the RCA mixture designs, a baseline life cycle assessment will be done on the OPC mixture design with 50% and 100% replacement of the aggregate.

Table 6 - HVFA Mixture Designs

Mean f`c (psi)	Specified f`c (psi)	% Ash	Water (lb/yd^3)	Cement (lb/yd^3)	Ash (lb/yd^3)	Coarse Agg (lb/yd^3)	Fines (lb/yd^3)
4500*	4000	40	308.4	406.2	270.8	1863	920.5
4500*	4000	80	308.4	135.4	541.6	1863	849.3
4100	3600	40	308.4	377.4	251.6	1863	965.9
4100	3600	80	308.4	125.8	503.2	1863	899.8
3900	3400	40	308.4	363.6	242.4	1863	987.7
3900	3400	80	308.4	121.2	484.8	1863	924
3050	2550	40	308.4	308.4	205.6	1863	1074.8
3050	2550	80	308.4	102.8	411.2	1863	1020.7

*Baseline cases without functional equivalence

Life Cycle Inventory Assessment

Results

The results of the Life cycle inventory assessment are presented in Tables 7 - 12. The data for carbon emissions are presented in mass of carbon dioxide equivalents per mass of material (kg/kg), energy usage is presented in MJ/kg and virgin aggregate is presented in lb/yd^3. The results are also presented graphically in Figures 10 - 12 for ease of comparison. The mixture designs in Figures 10 - 12 are identified by their replacement rate, the mixture material and the compressive strength in hundreds of psi. For each RCA mixture design, a replacement rate of 50% and 100% of the virgin aggregate was assessed. Detailed tables for the carbon emissions and energy use for each mixture design are presented in Appendix II.

Table 7 - RCA Carbon Emissions (CO_2e kg/kg)

COV	0% RCA	50 %RCA	100% RCA
0.1	0.1880	0.1936	0.1990
0.12	-	0.2170	0.2220
0.14	-	0.2370	0.2420
0.16	-	0.2650	0.2700

Table 8 - RCA Energy Usage (MJ/kg)

COV	0% RCA	50 %RCA	100% RCA
0.1	1.2570	1.9540	2.6512
0.12	-	2.0630	2.7380
0.14	-	2.1560	2.8120
0.16	-	2.2840	2.9150

Table 9 - RCA Virgin Aggregate Usage (lb/yd^3)

COV	0% RCA	50 %RCA	100% RCA
0.1	2854.7	1923.2	991.7
0.12	-	1844.1	912.6
0.14	-	1776.0	844.5
0.16	-	1680.9	749.4

Table 10 - HVFA Carbon Emissions (CO_2e kg/kg)

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	0.1880	0.1191	0.0474
0.8	-	0.1120	0.0450
0.6	-	0.1080	0.0440
0.4	-	0.0930	0.0390

Table 11 - HVFA Energy Usage (MJ/kg)

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	1.2570	0.9316	0.5449
0.8	-	0.8860	0.5260
0.6	-	0.8640	0.5170
0.4	-	0.7770	0.4820

Table 12 - HVFA Virgin Aggregate Usage (lb/yd^3)

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	2854.7	2783.5	2712.3
0.8	-	2828.9	2762.8
0.6	-	2850.7	2787.0
0.4	-	2937.8	2883.7

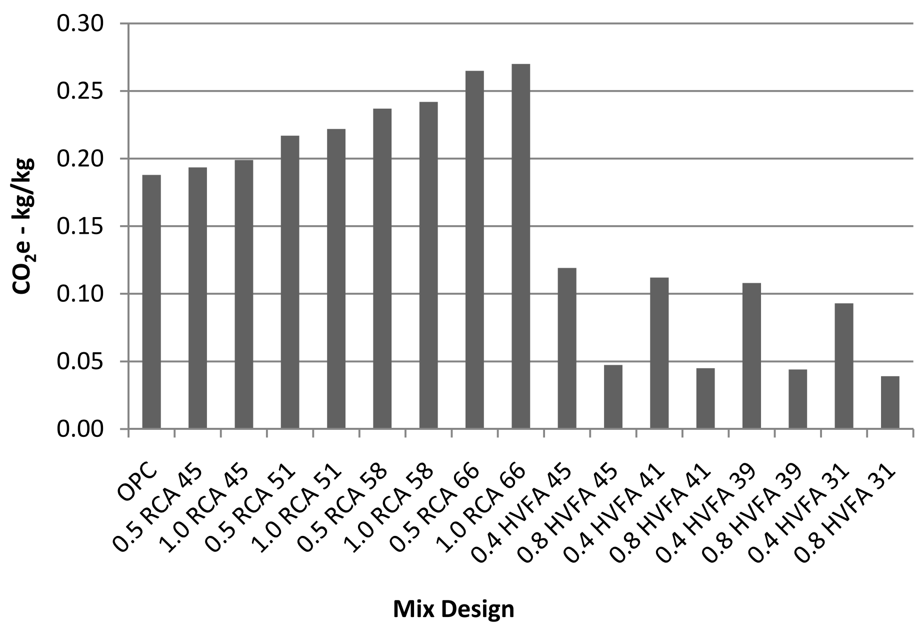

Figure 10 - Carbon Emissions

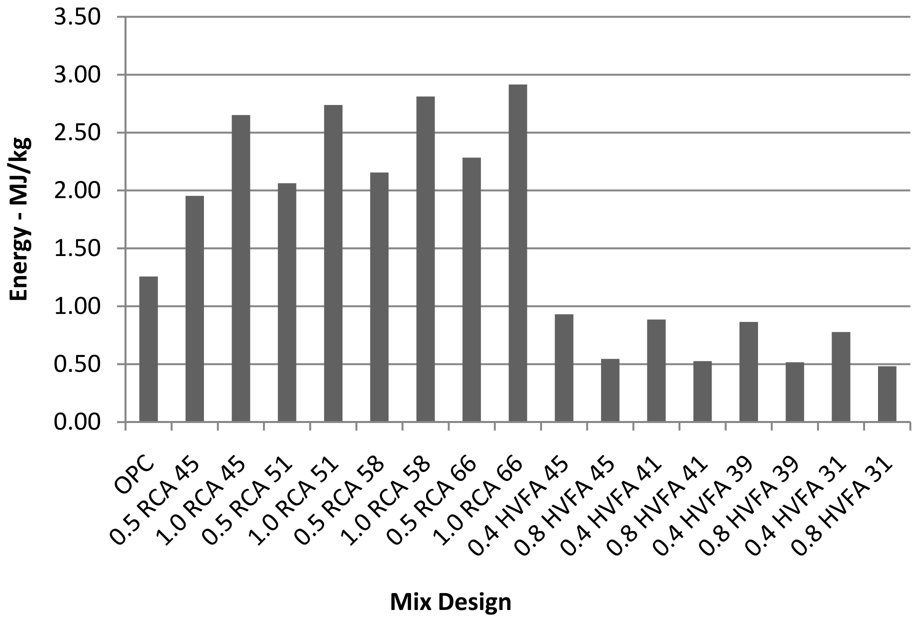

Figure 11 - Energy Use

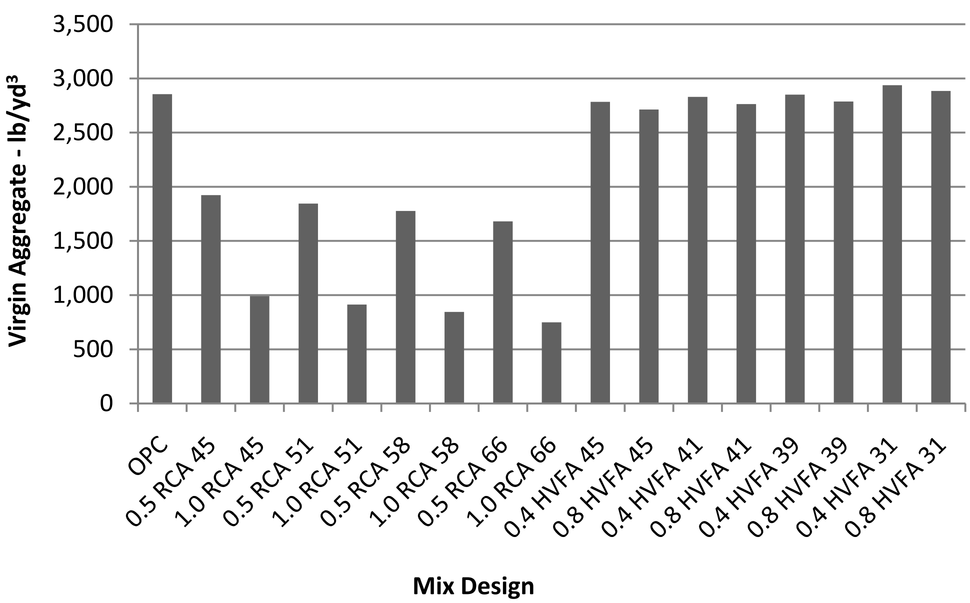

Figure 12 - Virgin Aggregate Use

Ordinary Portland Cement

For the 17 different mixture designs compared in the results section the ordinary portland cement was the seventh best mixture design for both CO_2 emissions and energy use. The ordinary portland cement mixture used more energy and CO_2 emissions than all the high volume fly ash mixture designs and less energy and CO_2 emissions than the recycled concrete aggregate mixture designs. The main reason the ordinary portland cement mixture design used more CO_2 and energy compared to the high volume fly ash mixture designs was due to cement production. Cement requires large amounts of CO_2 and energy to be produced (approximately 1 kg CO_2e/kg cement and 5.59 MJ/kg). With the reduction of cement required in the mixture, savings can be made in the environmental impact of the material.

Recycled Concrete Aggregate

The recycled concrete aggregate mixture designs perform the worst for CO_2 emissions and energy usage. The manufacturing process for recycled concrete aggregate uses approximately 0.0196 kg CO_2e/kg and 0.25 MJ/kg, compared to the virgin aggregate manufacturing process which uses 0.0052 kg CO_2e/kg and 0.083 MJ/kg. This is a 277% increase in the use of carbon and a 200% increase in the energy use. The statistics for energy use and carbon emission are for the heating and rubbing method. This method is not currently used in the United States, but the data is used because it is presented clearly and because this or a similar method to process the material will likely be required if RCA is to be used in structures. The heating and rubbing method heats the RCA to 300 degrees Celsius to dehydrate the cement paste, causing the paste to become brittle so that it can be rubbed off. This method requires large amounts of fuel and energy in order to achieve desired temperatures (Shima et al., 2005). One byproduct of the heating and rubbing method can be cement, which can be used in the mixture design and reduce the environmental impact of the recycled concrete aggregate. The possible cement production from this method was not taken into consideration for the life cycle inventory assessment.

Another issue with the production of recycled concrete aggregate is the transportation of the aggregate. The nearest facility to Astoria, OR that manufactures recycled concrete aggregate is located in Colton, CA. The distance the recycled aggregate must travel is around 1000 miles compared to a distance of around 20 miles required for the other aggregates to travel. The travel distance has a large effect on the overall life cycle inventory assessment values.

Finally due to the increase in the variability of the material strengths, the increase in required compressive strength requires more cement production. Since cement production is the largest contributor to CO_2 emissions and energy use, this has a large negative effect on the overall life cycle inventory assessment values.

The baseline case, the mixture design that does not achieve functional equivalence, also has an increase in carbon emissions and energy use compared to the OPC mixture design. Among the RCA mixture designs, the baseline case has the lowest carbon emissions and energy use. The trend of the RCA results show an increase in the carbon emissions and energy use as the compressive strength of the mixture design increases.

The positive environmental aspect of using recycled concrete aggregate is the reduction in use of virgin aggregate material and reduction of used concrete to be dumped in landfills.

High Volume Fly Ash

The high volume fly ash mixture designs outperform all other mixture designs. The increase in resistance to chloride diffusion allows for a decrease in the mean compressive strength of the mixture designs to achieve functional equivalence. This allows for a reduction of cement, which reduces the effect of the manufacturing process of the cement. In addition to the decrease in mean compressive strength, the allowance for the fly ash to partially replace the cement further decreases the negative effects of the cement manufacturing process. The baseline case also shows significant reduction in carbon emissions and energy use compared to the ordinary portland cement model. The carbon emissions and energy use slightly decrease as the compression strength of the mixture design decreases.

The only area in which the high volume fly ash increased the CO_2 emissions and the energy usage compared to the other two types of mixture designs was in the fly ash and cement transportation. The fly ash is required to be transported around 68 miles from Centrialia, WA to Seattle, WA and then the concrete travels from Seattle, which requires transportation of 175 miles. The large transportation distances negatively affects the carbon emissions and energy use for HVFA mixture designs. It is also important to consider weak early strength which is a property of high volume fly ash concrete, which can incur greater construction costs and increased difficulty during the construction.

Discussion & Conclusion

Discussion

Discussion of Methods

To better assess the effects of functional equivalence on the life cycle assessment data, the results are normalized using the baseline mixture designs that do not achieve functional equivalence. The normalized results are presented in Tables 13 - 18.

Table 13 - Normalized RCA Carbon Emissions

COV	0% RCA	50 %RCA	100% RCA
0.1	1.00	1.00	1.00
0.12	-	1.12	1.12
0.14	-	1.22	1.22
0.16	-	1.37	1.36

Table 14 - Normalized RCA Energy Usage

COV	0% RCA	50 %RCA	100% RCA
0.1	1.00	1.00	1.00
0.12	-	1.06	1.03
0.14	-	1.10	1.06
0.16	-	1.17	1.10

Table 15 - Normalized RCA Virgin Aggregate Usage

COV	0% RCA	50 %RCA	100% RCA
0.1	1.00	1.00	1.00
0.12	-	0.96	0.92
0.14	-	0.92	0.85
0.16	-	0.87	0.76

Table 16 - Normalized HVFA Carbon Emissions

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	1.00	1.00	1.00
0.8	-	0.94	0.95
0.6	-	0.91	0.93
0.4	-	0.78	0.82

Table 17 - Normalized HVFA Energy Usage

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	1.00	1.00	1.00
0.8	-	0.95	0.97
0.6	-	0.93	0.95
0.4	-	0.83	0.88

Table 18 - Normalized HVFA Virgin Aggregate Usage

Coeff	0% HVFA	40% HVFA	80% HVFA
1.0	1.00	1.00	1.00
0.8	-	1.02	1.02
0.6	-	1.02	1.03
0.4	-	1.06	1.06

The normalization of the results show that the baseline case for RCA underestimates the carbon emissions and energy usage of the mixture design, while the baseline case overestimates the use of virgin aggregate. The largest increase is seen in the carbon emissions for the RCA, which increases by 37% for a COV of 0.16. The normalization for the HVFA data shows the baseline case overestimates the carbon emissions and energy usage, while underestimating the virgin aggregate use. The largest decrease is seen in the carbon emissions for HVFA, which only uses 78% of the baseline case for the 40% HVFA mixture design with a diffusion coefficient factor of 0.4.

Significant increases and decreases are shown in the life cycle inventory assessment data, which suggest that functional equivalence is needed to accurately compare life cycle inventory data and to assess the best options for decreasing environmental impact. Research efforts should be made to accurately assess the environmental impacts of different materials used in structures. The method presented in this thesis represents the beginning stages in developing an accurate method of assessing environmental impact of certain materials.

The computational time required to obtain these results is high, about two hours on a common computer for each mix design using FORM and about five days for each sampling result. The effort of achieving functional equivalence led to changes in concrete compressive strength. While useful in research, in practice the significant computation time makes this method unusable. To make this useful for practitioners, researchers need to present distilled and organized results.

Areas of Improvement

More research is required for probabilistic assessments of the environmental impacts of the different mixture designs. Throughout the literature, probabilistic distributions of concrete with ordinary portland cement have been well documented. The appropriate values for probabilistic distributions of concrete with recycled concrete aggregate are less known. Some researchers have determined that concrete with recycled concrete aggregate has a larger coefficient of variation compared to ordinary portland cement, while other researchers determined that the coefficient of variations are the same. The data for recycled concrete aggregate is also not as useful for modeling in-situ compressive strengths, due to the fact that all the tests for recycled concrete aggregate have been done under laboratory conditions.

In this work only one time history was used to represent a Cascadia Subduction Zone interplate earthquake. Earthquake time histories have many different properties including frequency, duration and peak acceleration. All of these can vary widely from one time history to the next. To better represent the hazard of the Cascadia Subduction Zone, a wide variety of time histories should be used to better represent the possible array of earthquakes. These earthquake time histories should represent the possibility of a interplate rupture of the Cascadia Subduction Zone occurring, a intraplate earthquake occurring in the subducting Juan de Fuca plate, and the possibility of a crustal earthquake occurring in the North American plate. These different types of earthquakes all have different probabilities of occurring in Astoria, OR and should be represented with different probability of occurrences in the modeling.

Failure is defined in this project as strain exceeding 0.004 in the columns. There are several more performance states that could provide additional information to the model. Russell-Smith and Lepech (2009) chose five different performance states: no damage, slight damage, moderate damage, extensive damage and complete collapse. With multiple performance states, the recurring embodied energy can be more accurately accounted for, since repair of each damage state would need to be assessed. In the case of complete collapse a entire new bridge structure would need to be constructed, and the initial embodied energy would need to be assessed.

Mixture designs should be improved by incorporating all the different material requirements of the mixture designs. Concrete with recycled concrete aggregate requires plasticizers and additional water to improve workability of the mixture. High volume fly ash concrete mixture designs also can require plasticizers to achieve desired workability. The use of air entrained admixtures can be required when frost resistance is necessary, but the effectiveness of air-entrainers decrease with the increase of fly ash (Haque et al 1984).

While using the FORM analysis to determine the failure point of the ordinary portland cement mixture design, the analysis determined a large reliability index of 6.24. Such a large reliability index means that the failure point is well into the tail end of the distribution. This makes the model very sensitive to the selection of probabilistic distribution. Adding other sources of uncertainty will bring the design point away from the tail of the distribution leading to potentially more physically meaningful conclusions.

Recycled concrete aggregate is not commonly used in structures, so there is no available data on the manufacturing process that would be required. The heating and rubbing method was used, which is used in Europe and Japan to reuse concrete as aggregate. If recycled concrete aggregate is used in structure, the appropriate manufacturing process for the material should be used in the LCA process, which will provide more accurate results.

Conclusion

To reduce the environmental impact structures have on the environment, specific goals should be set to reduce impacts such as carbon emissions, energy usage and virgin aggregate usage. Such specific goals can only be assessed with accurate measurement standards. Simplistic goals can also have negative environmental impact, e.g. in reducing carbon emissions, virgin aggregate production could increase. Systems that do not measure the environmental impact of materials used can reduce a structure's environmental impact in one area, but cannot assure that environmental goals are being achieved.

Provided is a general method to determine the environmental impact of different materials while taking into account probabilistic properties and their effect on functional equivalence. Functional equivalence is set as the seismic resilience of a bridge structure over a 75 year service life. The method shows the effect of functional equivalence on life cycle assessment data, with maximum increases and decreases in carbon emissions of 37% and 22% respectively (over the baseline case that does not consider function equivalence in this way). The method has been simplified to determine the usefulness of this approach. More research is required to draw more realistic conclusions about actual environmental impacts of the concrete mixtures considered in this study.

The results of the life cycle inventory assessment highlight the unintuitive nature of decisions regarding sustainable development. For example, using recycled concrete aggregate would seem to have less environmental impact than using virgin aggregate. However, with the addition of the manufacturing process and the uncertainty of the material, it appears that using recycled concrete aggregate in the specific situation presented is not the best solution. There are many viable uses for recycled concrete aggregate such as fill for roadways, but this thesis suggests the use of the material in structures has an adverse impact on carbon emissions and the energy used. On the other hand, using high volume fly ash cement appears to reduce the same impacts. Using a high volume fly ash mixture design helps reuse fly ash (waste left behind by the combustion process), helps reduce the embodied energy of a structure, and leads to a reliable concrete material.

This study argues that decisions regarding sustainable development can be unintuitive and that simplistic environmental goals can have adverse consequences. Specific to structures, the results demonstrate the importance of considering natural hazards and durability issues during the life span of the structure. In the long term it is hoped that decisions regarding sustainability of structures are made systematically and accurately, either using the techniques used in this thesis or other techniques yet to be developed.

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Appendices

Appendix I - OpenSees Model

```
#units are inches and pounds

#Open Model Builder
wipe
model basic -ndm 2 -ndf 3

# Set up nodes
#    #    x      y
node 1  102      0
node 2  510      0
node 3    0  190.8
node 4  102  190.8
node 5  510  190.8
node 6  612  190.8

#Fix nodes
#   #  DX  DY  RZ
fix 1   1   1   1
fix 2   1   1   1
fix 3   0   1   0
fix 6   0   1   0

#Define Nodal Masses
#    n#    Mx    My   Mz
mass  1     9     9 1e-9
mass  2     9     9 1e-9
mass  3    44    44 1e-9
mass  4   228   228 1e-9
mass  5   228   228 1e-9
mass  6    44    44 1e-9

#DEFINE MATERIALS
# define geometric transformation
geomTransf Linear 1

# Confined Concrete
set fc1C [expr 1.3*($CompS)]
set Ecc [expr 57000*sqrt($fc1C)]
set eps1C [expr 2*$fc1C/$Ecc]
set eps2C [expr 5*$eps1C]
# Unconfined Concrete
set Ec [expr 57000*sqrt($CompS)]
set eps1U [expr 2*$CompS/$Ec]
set fc2U [expr 0.20*$CompS]
set eps2U [expr 5*$eps1U]
#                         tag      f`c    strain   crushing  strain
uniaxialMaterial Concrete01 1  -$fc1C     -$eps1C   -$CompS  -$eps2C
uniaxialMaterial Concrete01 2  -$CompS    -$eps1U    -$fc2U  -$eps2U

#Steel
uniaxialMaterial Steel01    3  60000    29000000    0.014
#Define Sections
#Beam Section
section fiberSec 1 {
```

```
#           mat tag   # div. Y   # div. Z   yI   zI  yJ  zJ
  patch rect      2         14         32   -7  -40   7  -8
  patch rect      2         84          8  -42   -8  42   0
#                  mat tag nbars  area     ys     zs     ye    ze
  layer straight       3       3   1.56   -3.8  -34.8   3.8  -34.8
  layer straight       3       3   1.56   -3.8  -30.8   3.8  -30.8
  layer straight       3       3   1.56 -12.75   -4.2 12.75  -4.2
  layer straight       3       3   1.56   -3.8     -8   3.8     -8}

#Column Section
section fiberSec 2 {
  patch rect 1 15 15 -7.5 -7.5  7.5  7.5
  patch rect 2  5 20  -10  -10 -7.5   10
  patch rect 2  5 20  7.5  -10   10   10
  patch rect 2 15  5 -7.5  -10  7.5 -7.5
  patch rect 2 15  5 -7.5  7.5  7.5   10
  layer straight 3 3 0.79 -7.5 -7.5 7.5 -7.5
  layer straight 3 3 0.79 -7.5  7.5 7.5  7.5
  layer straight 3 2 0.79 -7.5    0 7.5    0}


#DEFINE DISPLACEMENT BEAM-COLUMN ELEMENTS
#                      tag  Inode  Jnode   #Integ P  SecTag  transf
element dispBeamColumn   1      1      4         4       2       1
element dispBeamColumn   2      2      5         4       2       1
element dispBeamColumn   3      3      4         4       1       1
element dispBeamColumn   4      4      5         4       1       1
element dispBeamColumn   5      5      6         4       1       1

#Define Gravity
pattern Plain 1 Linear {
  load 1 0 -3310 0 -const
  load 2 0 -3310 0 -const
  load 3 0 -16900 0 -const
  load 4 0 -84600 0 -const
  load 5 0 -84600 0 -const
  load 6 0 -16900 0 -const}

# Stress Strain Recorders
recorder EnvelopeElement -file strain3.txt -ele 1 section 1 fiber 7.5 0
stressStrain
recorder EnvelopeElement -file strain4.txt -ele 1 section 1 fiber -7.5 0
stressStrain

# DYNAMIC ground-motion analysis -------------------------------------------
set accelSeries "Series -dt 0.01 -filePath ModifiedTokachi.txt -factor .394"
pattern UniformExcitation 2 1 -accel $accelSeries
rayleigh 0.1137 0. 0. 0.0011

# Set Time steps and runtime
set dt 0.02
set runtime 40


#FEA Analysis Commands
constraints Plain
numberer RCM
```

```
system BandSPD
test EnergyIncr 1.0e-3 10
algorithm Newton
integrator Newmark 0.5 0.25
analysis Transient
analyze [expr int($runtime/$dt)] $dt
```

Appendix II - Life Cycle Inventory Assessment Data

The units for CO_2 emissions are in mass of CO_2 equivalent per mass of material (kg/kg) and the units for energy use are MJ/kg.

Table A1 - Ordinary Portland Cement Mixture Design

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.87E-03	0.0617
Aggregate Transportation	2.94E-06	0.0236
Cement Production	1.78E-01	0.9859
Cement Transportation	6.98E-07	0.0056
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.188	1.257

Table A2 - 50% RCA with 4500 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	1.93E-03	0.0309
Recycled Aggregate Manufacture	7.29E-03	0.0929
Aggregate Transportation	8.20E-05	0.6588
Cement Production	1.78E-01	0.9859
Cement Transportation	6.98E-07	0.0056
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.194	1.954

Table A3 - 100% RCA with 4500 psi compressive strength

	CO_2 Emissions	Energy Use
Recycled Aggregate Manufacture	1.46E-02	0.1858
Aggregate Transportation	1.61E-04	1.2939
Cement Production	1.78E-01	0.9859
Cement Transportation	6.98E-07	0.0056
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.199	2.651

Table A4 - 50% RCA with 5100 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	1.87E-03	0.0299
Recycled Aggregate Manufacture	7.06E-03	0.0900
Aggregate Transportation	7.95E-05	0.6380
Cement Production	2.02E-01	1.1184
Cement Transportation	7.92E-07	0.0064
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.217	2.063

Table A5 - 100% RCA with 5100 psi compressive strength

	CO_2 Emissions	Energy Use
Recycled Aggregate Manufacture	1.41E-02	0.1800
Aggregate Transportation	1.56E-04	1.2532
Cement Production	2.02E-01	1.1184
Cement Transportation	7.92E-07	0.0064
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.222	2.738

Table A6 - 50% RCA with 5800 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	1.82E-03	0.0290
Recycled Aggregate Manufacture	6.86E-03	0.0875
Aggregate Transportation	7.72E-05	0.6203
Cement Production	2.22E-01	1.2318
Cement Transportation	8.72E-07	0.0070
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.237	2.156

Table A7 - 100% RCA with 5800 psi compressive strength

	CO_2 Emissions	Energy Use
Recycled Aggregate Manufacture	1.37E-02	0.1750
Aggregate Transportation	1.52E-04	1.2184
Cement Production	2.22E-01	1.2318
Cement Transportation	8.72E-07	0.0070
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.242	2.812

Table A8 - 50% RCA with 6550 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	1.75E-03	0.0279
Recycled Aggregate Manufacture	6.59E-03	0.0840
Aggregate Transportation	7.42E-05	0.5958
Cement Production	2.50E-01	1.3887
Cement Transportation	9.83E-07	0.0079
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.265	2.284

Table A9 - 100% RCA with 6550 psi compressive strength

	CO_2 Emissions	Energy Use
Recycled Aggregate Manufacture	1.32E-02	0.1681
Aggregate Transportation	1.46E-04	1.1701
Cement Production	2.50E-01	1.3887
Cement Transportation	9.83E-07	0.0079
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.270	2.915

Table A10 - 40% HVFA with 4500 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.84E-03	0.0613
Aggregate Transportation	2.92E-06	0.0235
Fly Ash Transportation	1.30E-06	0.0104
Cement Production	1.09E-01	0.6027
Cement Transportation	6.69E-06	0.0537
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.119	0.932

Table A11 - 80% HVFA with 4500 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.81E-03	0.0609
Aggregate Transportation	2.90E-06	0.0233
Fly Ash Transportation	2.65E-06	0.0213
Cement Production	3.69E-02	0.2048
Cement Transportation	6.82E-06	0.0547
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.047	0.545

Table A12 - 40% HVFA with 4100 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.91E-03	0.0623
Aggregate Transportation	2.97E-06	0.0239
Fly Ash Transportation	1.21E-06	0.0097
Cement Production	1.01E-01	0.5603
Cement Transportation	6.22E-06	0.0499
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.112	0.886

Table A13 - 80% HVFA with 4100 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.88E-03	0.0620
Aggregate Transportation	2.96E-06	0.0237
Fly Ash Transportation	2.46E-06	0.0197
Cement Production	3.43E-02	0.1901
Cement Transportation	6.33E-06	0.0508
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.045	0.526

Table A14 - 40% HVFA with 3900 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.94E-03	0.0628
Aggregate Transportation	3.00E-06	0.0241
Fly Ash Transportation	1.16E-06	0.0093
Cement Production	9.73E-02	0.5400
Cement Transportation	5.99E-06	0.0481
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.108	0.864

Table A15 - 80% HVFA with 3900 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	3.92E-03	0.0625
Aggregate Transportation	2.98E-06	0.0239
Fly Ash Transportation	2.37E-06	0.0190
Cement Production	3.30E-02	0.1831
Cement Transportation	6.09E-06	0.0489
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.044	0.517

Table A16 - 40% HVFA with 3050 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	4.06E-03	0.0648
Aggregate Transportation	3.09E-06	0.0248
Fly Ash Transportation	9.89E-07	0.0079
Cement Production	8.26E-02	0.4586
Cement Transportation	5.09E-06	0.0409
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.093	0.777

Table A17 - 80% HVFA with 3050 psi compressive strength

	CO_2 Emissions	Energy Use
Virgin Aggregate Manufacture	4.05E-03	0.0646
Aggregate Transportation	3.08E-06	0.0247
Fly Ash Transportation	2.01E-06	0.0161
Cement Production	2.79E-02	0.1551
Cement Transportation	5.16E-06	0.0415
Concrete Manufacturing	6.63E-03	0.0178
Concrete Transportation	2.02E-05	0.1622
Total	0.039	0.482