STRUCTURAL OPTIMIZATION AND LIFE-CYCLE SUSTAINABILITY ASSESSMENT OF REINFORCED CONCRETE BUILDINGS IN SEISMIC REGIONS

A Thesis

Presented to

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In Partial Fulfillment

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Master of Science

in Civil and Environmental Engineering

by

Kazi Ashfaq Hossain

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ABSTRACT

Recent devastating natural hazards worldwide have underscored hazard resilience as an important component of sustainability. This thesis presents a comprehensive sustainability quantification methodology for reinforced concrete (RC) buildings subjected to earthquakes by developing and using a new life-cycle assessment (LCA) framework. The sustainability components: cost associated with various stages (economic aspects), environmental emissions and waste generations (environmental aspects), and downtime (social aspects), are quantified as a part of this LCA framework. Although initial construction phase has a significant contribution to cost and environmental impact, future structural performance plays an important role by affecting repair cost, environmental impact due to repair activity, and death and downtime due to unsatisfactory performance. A structural optimization problem is introduced within the sustainability assessment framework for achieving optimality in seismic design. The proposed approach is novel because it incorporates all essential components of sustainability: economy, environment and society within the same framework, and because it assigns, for the first time, these components as performance objectives in order to obtain optimality in life-cycle performance. The developed sustainability assessment framework and structural optimization methodology is applied to a case study building to illustrate the benefits of the proposed approach in reducing the lifetime impacts of RC buildings subjected to earthquakes.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

The motivation for the research presented in this thesis is to reduce the devastating consequences of seismic hazards on civil infrastructure by reducing cost, environmental and social impact, which are the three fundamental components of sustainability, through optimal seismic design. Details are described in this chapter.

1.1 Preamble

Recent earthquake events all over the world have shown that there is still a need of a significant amount of research in order to reduce the catastrophic impacts of earthquake on the society. While the amount of impact is generally scenario-specific and depends on several factors, the general consequences range from economic loss to fatalities, injuries, and homelessness among others. Civil infrastructure are traditionally designed and constructed to provide adequate strength in order to survive under extreme loading. This is achieved through compliance with regulatory documents, and it is anticipated that the direct and indirect effects of earthquakes on society will be minimized. After the Northridge earthquake (1994), it was observed that compliance with existing seismic codes in the United States resulted in proper safety of life, but could not prevent structures from experiencing an inordinate amount of damage. The cost associated with seismic damage led to severe economic repercussions, which encouraged practitioners to move towards performance-based design codes rather than serviceability and strength-based design. However, recent devastating seismic hazards worldwide, especially those in Chile (2010) and Christchurch (2011), have also demonstrated unexpected structural performance of buildings which shows that more concentrated efforts towards seismic damage minimization is necessary.

The ultimate objective of all engineering interventions is the betterment of human lives at personal, social, and global levels. Thus any engineering effort is fundamentally in line with the goal of sustainability, which revolves around minimizing the impacts on three interdependent components: economy, environment, and society. Damage due to unsatisfactory structural performance possesses considerably high economic, environmental and social impacts and act as a threat to achieving the goals of sustainability. The cost of repairing, retrofitting, or reconstructing damaged infrastructure is significantly high. These repair activities also affect the environment by consuming high amount of natural resources and energy, and by generating harmful substances and construction debris. Although the death toll and injuries have been reduced through improved practices, the number has not yet reached an acceptable level. Due to these reasons, seismic events possess great implications on the sustainability in construction industry and needs to be properly addressed by engineers and decision makers.

1.2 Problem Statement

In order to ensure sustainable development, sustainability needs to be properly evaluated in a seismic context and more broadly in the context of natural hazards. This requires a comprehensive knowledge regarding the interrelation between various aspects of seismic hazards, structural performance and sustainability components in order to move towards a sustainable solution. Therefore, efforts are required to properly define sustainability and its components in a seismic scenario. Afterwards earthquake damage should be quantified and defined in terms of sustainability components. Seismic-resistant design should incorporate sustainability components as performance objectives for ensuring seismic sustainability of the proposed design.

1.3 Objectives and Scope

This thesis aims to address the concerns presented in the previous section. Therefore, the objective is to quantify sustainability in terms of cost, environmental and social impacts for seismic assessment of buildings. A considerable amount of research has been conducted focusing on some of the sustainability components individually as discussed below in Chapter 2. However, a complete sustainability assessment and sustainable design approach requires consideration of all three components; thus the novelty of this research lies in the integration of these sustainability objectives under the same framework.

This study introduces a comprehensive probabilistic framework for seismic sustainability assessment of reinforced concrete (RC) buildings with due consideration to three primary components: cost, and environmental and social impact. Life-cycle assessment (LCA) methodology is developed by properly integrating the three interrelated functions: life-cycle structural performance assessment (LCSPA), life-cost assessment (LCCA), and life-cycle environmental impact assessment (LCEIA). The proposed framework evaluates seismic hazard, structural response, and structural damage state in a probabilistic manner using LCSPA considering uncertainties at each step. Cost, environmental impact and time inventories corresponding to each life-cycle phases are developed, and the impacts are evaluated through LCCA and LCEIA. The other uniqueness of this approach is that it converts probabilistic structural damage due to natural hazards to quantifiable economic, environmental and social impacts that is amenable to transparent decision making.

Another distinctive feature of this research is the enhancement of seismic design through incorporation of the quantifiable impacts mentioned above in a multi-objective problem for reducing them. This is done by selecting sustainability components as performance objectives for seismic resistant designs. This allows selection of design variables which produces the minimum life-cycle impact in terms of cost, environmental and social impact. Although the framework is developed for RC buildings under seismic hazard, it can be properly tuned and applied to other structural systems under different hazard scenarios.

1.4 Organization of the Thesis

Chapter 2 focuses on providing the background information about the subject matter. It first introduces the concept of sustainability and the importance of comprehensive sustainability assessment methodology for construction industry to properly address seismic effects. Then reviews of different LCA studies are presented. Finally different optimization techniques along with their application in optimal structural seismic design are discussed in detail.

Chapter 3 outlines the framework for seismic sustainability assessment with due consideration of all sustainability components. LCSPA is introduced to evaluate structural performance in terms of probable structural damage from future earthquakes. The damage is then converted into cost, environmental impact, and quantifiable social impact using inventories. Finally, a multi-level multi-objective optimization methodology is developed to find the optimally sustainable solutions.

Chapter 4 provides an application of the framework developed in Chapter 3 on a case study RC building frame. Successive completion of hazard, structural, and damage analysis provides probabilistic damage values of structural components. Afterwards by using life-cycle functions those damage values are converted to total cost, total environmental impact, and social impact (in terms of construction and repair time). Finally, the optimization technique is applied to investigate the selection of design parameters on the sustainability of the structure.

Chapter 5 concludes the thesis with a summary of the findings of the research. Suggestions for future research are also given here.

2. BACKGROUND AND LITERATURE REVIEW

In Chapter 2, sustainability concept and its components are first introduced, and the importance of sustainable development with respect to building industry is discussed. Then, the necessity of a comprehensive sustainability assessment framework of structures subjected to seismic events is described. LCA is introduced as a technique to quantify sustainability through three interactive functions: i) life-cycle structural performance assessment, ii) life-cycle cost assessment, and iii) life-cycle environmental impact assessment. A complete review of LCA studies considering these three functions are also presented in this chapter. Finally, different optimization techniques and their applications in seismic design of RC structures are discussed.

2.1 Sustainable Development in Civil Infrastructure

The construction industry has been associated with a considerable amount of harmful gases that contributes to environmental pollution (Orabi et al. 2012). The increased rate and growing volume of pollutants caused by numerous development initiatives are likely to have increased the carbon footprint and other environmental impacts in recent years. These impacts are suspected to instigate various natural disasters (NASA 2013). In addition to these, all construction projects are associated with high consumption of raw materials and non-renewable energy. The rapidly increasing rate of resource exhaustion and environmental impact has prompted a movement towards sustainable approach in design, construction, and operation of various products and processes. As a result, ensuring sustainability in any development project is, now-a-days, considered to be an essential factor for the general welfare and continuous advancement of the society. Construction industry has embraced this concept and has undergone major changes in fully achieving sustainable development. At the same, construction industry is greatly responsible for exhaustion of raw materials, consumption of renewable and non-renewable energy, and generation of waste and harmful by-products in the form of harmful gases, liquid effluents, and solid wastes. Raw material production, transportation of raw materials from factories to site, and operation of on-site machineries during construction process release enormous amount of greenhouse gases (GHG). Operation phase uses significant amount of non-renewable energy in the form of HVAC which causes deterioration of environment. Buildings sector alone contributes to 40% of global raw stone, gravel, and sand consumption as well as exhaustion of 25% of virgin wood. Each year, buildings also use 40% of the energy and 16% of the water worldwide, mostly due to operational activities (Roodman et al. 1995). Construction activities around the world also contribute to as much as 40-50% of global GHG emission (California Integrated Waste Management Board 2000). These numbers do not comply with the sustainability goals that aim at achieving perfect balance among the environmental, economic and social dimensions.

Concrete is one of the most used materials, second only to water, in addition to being the largest consumed construction material in the world. The total amount of concrete and steel produced in 2007 all over the world is over 14 billion tonnes (Aitcin and Mindess 2011). The production of one tonne of cement emits as much as 927 kg of CO_2 which is the primary reason for global warming. A study shows that, a likely 5 billion tonnes of Portland cement will be produced in the year 2020 which will double the current level of CO_2 emission corresponding to cement production (Naik 2008). The US cement industry contributes to 1.5% of total US CO_2 emissions. According to Department of Energy, cement and steel production account for 0.33% and 1.8% of total annual energy consumption in the US (NRMCA 2012). Also, the US alone produces 140 million tonnes of debris associated with construction and demolition activities (Solid Waste Digest 2011). These numbers may apparently seem small compared to some other energy intensive activities such as heating and cooling services of residential and commercial facilities, transportation, and industrial operations. Nevertheless, the cumulative contribution of thousands of new construction projects every year is enormous. This results in increased environmental and economic impact on the society, which can be brought down through the employment of sustainable initiatives as presented in this thesis.

2.1.1 Sustainability Components

Sustainability first emerged with the goal of enhancing and improving the management process of natural resources and energy, thus conserving them both locally and globally. The initial goal was to maintain the natural ecosystem by reducing the impact on the environment by undertaking several initiatives. Though this concept was originally used for biological and human systems, it was later adopted by other sectors as well. The issue of sustainability has been present since the 50's when the early environmental movement started. The term sustainable development was coined in 'Our Common Future' a report of the World Commission on Environment and Development (WCED), popularly known as 'the Brundtland Report' (Brundtland 1987). The generalized definition, which is applicable for all technical disciplines, was given in this report as 'management of resources such that current generations are able to meet their needs without affecting the ability of future generations to meet their needs'. Sustainability has also been defined as 'improving the quality of human life while living within the carrying capacity of supporting eco-systems' (IUCN/UNEP/WWF 1991). However, practical interpretations of these definitions are rare.

In terms of environmental usage, sustainability mostly refers to the endurance of the ecological environment through minimization of environmental impact and effective conservation of natural resources. Environmental issues were the primary concern during the initial stage of the sustainability movement. Until now, sustainable development is often misinterpreted by referring to only 'green' or environment-friendly initiatives. As much as ensuring 'greenness' of products and services is important, other issues concerning sustainability such as adopting innovative techniques to reduce material usage (and subsequently cutting down carbon footprint), and future losses in the case of natural hazards are also essential. Hence, sustainability is now represented in terms of three interdependent, coherent, and mutually complementing components: economy, environment, and society. The goal of sustainability initiatives can be deemed as economic development, environmental protection, and social development, and is well comprehended from the triple-bottom line definition (see Figure 1) (Willard 2002).

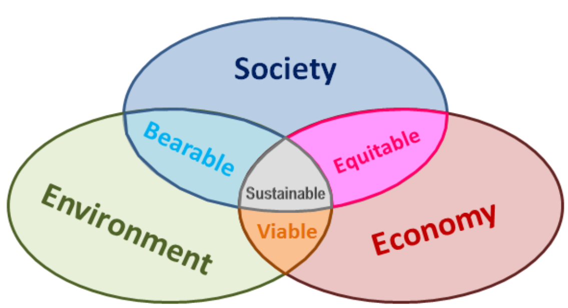

Figure 1: The triple bottom line of sustainability

Buildings are fundamentally designed from strength and serviceability point-of-view. As a result, reduction of environmental impacts through energy and material optimization is not usually considered as a design objective in building industry. The omission of sustainable design initiatives has vital implications at economic, environmental, and social levels. The impacts can easily be reduced by employing advanced sustainable solutions, novel design and construction practices. For building industry, some of the sustainability objective can be fulfilled through optimum material usage, use of eco-friendly and recyclable materials, incorporation of sustainable construction techniques, increasing product longevity, energy efficient designs among many others.

The importance of sustainable development has also been acknowledged by American Society of Civil Engineers (ASCE). The main motivation is due to the realization of the fact that materials and energy are limited, and construction sector consumes a massive portion of the remaining available resources. Hence, sustainable development was included as one of the seven fundamental canons of the code of ethics (ASCE 2009a). The society defines sustainable development as 'the process of applying natural, human, and economic resources to enhance the safety, welfare, and quality of life for all of society while maintaining the availability of the remaining natural resources.' (ASCE 2009a). Many of the other policies of ASCE require actions towards sustainable development; most crucial ones being: Policy 360 - Impact of Global Climate Change (ASCE 1990), Policy 418 - The Role of the Civil Engineer in Sustainable Development (ASCE 1993), Policy 488 - Greenhouse Gases (ASCE 2001), and Policy 517 - Millennium Development Goals (ASCE 2000). ASCE believes that civil engineers will be the leaders in achieving Vision 2025 that is designed to 'create a sustainable world and enhance the global quality of life' (ASCE 2009b). The ASCE Task Committee on Sustainable Design (TCSD), formed in 2009, has been assigned different tasks that include developing an action plan for the society to advance the principles of sustainable development in civil infrastructures. Although these policies promote and ensure sustainable practices within the industry, there still exist several issues related to the structural aspects of buildings that need to be properly addressed for a sustainable development.

2.1.2 Sustainability and Seismic Resilience

From a structural point-of-view, the first and foremost objective of any structure is that it would remain functional throughout its service life with minimum disruption to external disturbance. Buildings are fundamentally designed to demonstrate resilience against extreme loads. Structural resilience can be defined as the ability to accommodate sudden or abrupt forces a structure may experience, that is to protect itself from complete collapse, and to keep the structural damage to an accepted level through adaptability and resistance, thereby, accommodating enhanced and increased service life.

Resilience and sustainability are technically not the same concept because resilience concerns with adequate structural performance against sudden or sustained loads, whereas sustainability aims at balancing the economic, environmental and social factors. However, under seismic hazards, resilience directly influences cost, environmental and social impacts, which are the basic sustainability components of a structure. Hence, sustainable structures must have adequate resilience in order to maintain minimum damage in case of extreme events; and one cannot think of sustainability alone without resilience in the presence of natural hazards. Otherwise, a “sustainable” structure might undergo extensive damage or even collapse in extreme situation as a consequence of unsatisfactory structural performance. This will lead to substantial life-cycle cost for repair, retrofit, restoration or even complete replacement of elements or rebuilding of the whole structure. These repair activities also consume a considerable amount natural resources and nonrenewable energy, cause environmental impacts through emission of harmful substances and generation of debris. The social impact associated with natural hazards can be downtime to repair structures, relocation of affected population and death and injuries of people. For example, the number of fatalities from seismic events was close to 20,000 in 2011, while the total monetary loss due to earthquake related hazards was $500 to $750 billion (Daniell and Vervaeck 2012). This alarmingly high number of affected people and damage of infrastructure have brought the attention to resilient design practices.

In order to address these issues, Federal Emergency Management Agency (FEMA) suggested disaster resilience to be incorporated directly in the decision-making process for sustainable development (FEMA 2000). ASCE also promoted the linkage between these two concepts in roundtables titled ‘sustainability and resilience in infrastructure to protect the natural environment and withstand natural and man-made hazards’(ASCE 2009b). Some recent articles also featured disaster resilience as a means of achieving the sustainability goal (Lascher 2012; Nambier 2012). Therefore, it is a necessity to obtain an optimal solution between social, economic and environmental impact, and resilience for structures in hazard-prone regions. Since civil infrastructures are usually designed for longer lifetimes, life-cycle structural performance should be taken into account during sustainability assessment with the incorporation of risks from hazard.

For sustainability assessment, some advanced tools are currently being developed and implemented by different agencies. The most popular assessment methods in this line are Leadership in Energy & Environmental Design (LEED), Building Research Establishment Environmental Assessment Method (BREEAM), and Green Building Challenge (GBTool). These 'Green Building' rating systems evaluate the system level sustainability of buildings mostly based on innovation, material usage, and energy efficiency. For example LEED, which is developed by the United States Green Building Council (USGBC), have five major credit categories for new construction and major renovations of building projects: sustainable sites, water efficiency, energy and atmostphere, material and resouces, indoor environmental quality, along with two additional categories: innovation and design process, and regional priority credits. The structural aspect of a building, which should be the primary concern of building under natural hazards, is not explicitly offered any points. Some structural issues can be addressed under 'material and resources' and 'innovation in design' categories. However, these are only limited to steps taken towards reduction of embodied energy and life-cycle material usage.

'Green Buildings' are not always designed to exhibit resilient features. Recently, an LEED certified building in Oregon (Cheatham 2010) was observed to have structural issues such as cracked walls and ceilings, and buckling of post-tensioned concrete slab. While the primary reasons were noncompliance with the design code requirements and use of poor construction materials, the incident also pointed out the necessity of including structural aspect in sustainability assessment procedure. According to a report by Zolli (2012), New York City has the largest number of LEED-certified green buildings in the U.S . These buildings did not respond well during Hurricane Sandy. Though they were designed as sustainable structures expected to produce lower environmental impact, they were not resilient enough to sustain environmental loads. After the event, tons of debris is deposited and new construction materials are consumed for rebuilding, thus the buildings failed their initial objective, which was to produce lower lifetime environmental impact. To address this issue, experts suggest that LEED or any other Green Building assessment tool should recognize the regional context while developing credit systems; for example, assign points for seismic resistant features in the pacific coast, which is a high-seismic region. LEED is also looking forward to including more explicit credits for resilient features in the next few revisions.

In summary, adequate structural resistance and adaptability for minimization of damage and ability to quickly recover after a natural hazard should be incorporated as essential features for sustainable buildings, in addition to innovation and energy efficiency. Otherwise, the structure will become unusable long before the completion of its design life resulting in loss of capital, raw material and energy used for construction in addition to other indirect losses such as fatalities, injuries, and downtime. Therefore, comprehensive sustainability assessment techniques are necessary in order to properly incorporate structural response against future hazards which the structure may experience.

2.2 Life-Cycle Assessment for Sustainability Quantification

Sustainability needs to be properly represented in terms of quantifiable metrics in order to allow for comparisons and tradeoffs among alternative designs. Quantifying the sustainability of buildings is significantly more difficult compared to other products or processes due to reasons such as the presence of multiple materials, complicated manufacturing and construction processes, complex and changing functionality, long product design life in contrast with limited service life of components, constant interaction with users and environment, non-standardized processes, and insufficient data. In addition, development of sustainability quantification methodology needs proper understanding of different aspects of sustainability. Therefore, quantification of sustainability requires complete and systematic assessment of the three subcomponents (cost, environmental and social impacts), and the framework should be developed in such a way that it takes into account the whole life-span of the building. All structural systems are found to undergo distinct life-cycles comprising several stages such as material production, construction, operation or use, and disposal. As a result, LCA has become very popular among researchers and designers for sustainability evaluation of buildings.

2.2.1 LCA of Reinforced Concrete Buildings

LCA has recently gained popularity as a functional tool that assesses the performance of products and services in terms of energy use and other environmental impacts. This technique was originally proposed as a method to assess environmental performance of any product, process, or system from cradle-to-grave, starting from raw material extraction and manufacturing of material from the raw materials and ending with disposal of the materials to the earth (SETAC 1993). According to Scientific Applications International Corporation (SAIC), LCA is 'a technique to assess the environmental aspects and potential impacts associated with a product, process, or service'(SAIC 2006). Because of the guidelines provided by these agencies, the use of LCA tool is mostly restricted to the evaluation of interaction of the product with the environment at each of the interdependent life-cycle stages.

In this context, LCA is sometimes treated synonymously with environmental impact assessment. However, quantification of other sustainability components (cost and social impact) can also be explored through LCA. By accommodating economic and social aspects within the context of LCA has given rise to life-cycle sustainability impact from its original focus of life-cycle environmental impact assessment. Therefore in recent times, the scope of LCA has evolved into a much broader context from only environmental, ecological or health issues. It has turned into a systematic approach through which one can assess the effects of actions, deformations and interventions, such as use, maintenance, repair, retrofit, aging effects, on the structural performance over the total or rest of its service life (Liu and Frangopol 2006).

The International Organization for Standardization (ISO) outlined the fundamental principles and framework as well as the requirements and guidelines for LCA in ISO 14040 series (ISO 2006a; ISO 2006b). However, no technique or detailed procedure was recommended about how LCA should be performed. The general LCA methodology includes quantitative assessment of a material used, energy flows and environmental impacts of products throughout the product life (cradle-to-grave). The basic framework according to ISO 14040, shown in Figure 2, consists of four phases:

goal and scope definition; life cycle inventory analysis; life cycle impact assessment; and interpretation, each affecting the other phases in some way (ISO 2006a). Although the inventory and impact mentioned here refers to that of the environment, these interactive steps can be used to develop a generalized framework for sustainability assessment.

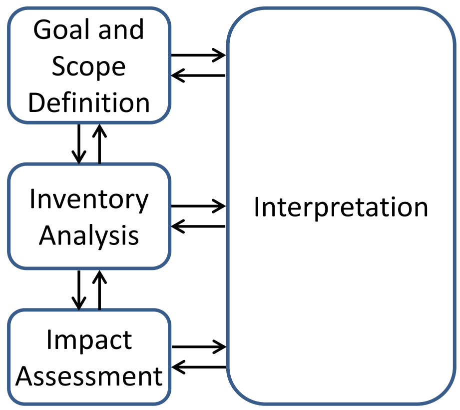

Figure 2: LCA according to ISO 14040 (ISO 2006a)

LCA is performed using several functions based on the objective of the study. The main focus of this study is sustainability assessment of RC buildings under earthquake hazard. Under such conditions, sustainability components are related to structural performance under future earthquake events. To address that, LCA needs to include a function, here referred to as life-cycle structural performance assessment (LCSPA), which takes into account the uncertainty in hazard, structural performance and the damage experienced by the structure. Then two other functions life-cycle cost assessment (LCCA) and life-cycle environmental impact assessment (LCEIA) are used to evaluate the total life-cycle cost and environmental impacts, respectively. There have been several studies regarding these individual LCA functions; however, very few of those addressed more than one in the same study. More specific applications of LCA functions in terms of structural performance, cost, and environmental impact are discussed in Section 2.2.2 through 2.2.4.

2.2.2 Life-Cycle Structural Performance Assessment Function of the Framework

LCSPA methodology evaluates the structural responses due to all forces which can occur during the service life of the structure. Traditionally, seismic structural analysis is performed deterministically, without consideration of uncertainties in the earthquake event, structural response, damage, and resulting losses. Typically the numerical model of a structure is subjected to a simplified code-based representation of earthquake forces. However, rapid development of computational tools has enabled using more advanced computational analysis methods such as nonlinear response history analysis and incremental dynamic analysis. Structural performance aims at relating structural demand with its capacity. The capacity is mainly governed by the strength, stiffness and ductility of the structure, whereas the seismic demand is site-specific, and depends on the type and number of faults in the vicinity of the structure along with the local soil condition. LCSPA aims at evaluating probable structural damage from the structural response indicators. Since the ground motion, structural responses, and the structural damage all possess uncertainties, it is more logical to express these in a probabilistic manner. Performance-based Earthquake Engineering (PBEE) methodology has been incorporated in this study for performing LCSPA. Hence, a review of related studies is presented the following.

The necessity of moving towards a performance-based approach was acknowledged in mid 90’s right after two massive earthquakes took place namely, Northridge (1994) and Kobe earthquakes (1995). The extent of structural damage and the resulting economic losses from these events were severe, although the structural designs were in well agreement with the seismic codes that were present at that time (Lee and Mosalam 2006). These incidents led to several publications such as Vision 2000 (SEAOC 1995) and FEMA 273 (FEMA 1997a) that pioneered the new design philosophy eventually formulating the PBEE methodology. The following section presents a brief review of PBEE methodologies.

PBEE can be defined as a system level structural performance assessment methodology subjected to seismic loads. In this methodology the different performance objectives are achieved when subjected to different seismic hazard levels through adopting different design, construction, and maintenance practices. As an example, the performance objective can be the threshold value of an action or a deformation based limit state or a damage state. Performance-based design is different from traditional design codes which only look forward to fulfilling the life-safety objective. The first introduction of performance based design can be traced back to the early twentieth century (Haselton and Deierlein 2007). However, the new documents provide much more detailed and comprehensive guidelines for performing performance seismic design and assessment.

The objective of Vision 2000 report (SEAOC 1995), one of the very first documents introducing the first generation PBEE, was to define a performance-based seismic design (PBSD) framework for buildings at different levels of seismic excitations. Various hazard levels were defined in terms of return periods or probabilities of exceedence (POE), while structural performance levels were defined as fully operational, operational, life safety, and collapse prevention. The designer was given the opportunity select the appropriate combination of hazard and performance levels depending on the building's occupancy, importance, and economic considerations. This relationship between performance level, hazard level and type of structure as recommended in Vision 2000 is shown in Figure 3.

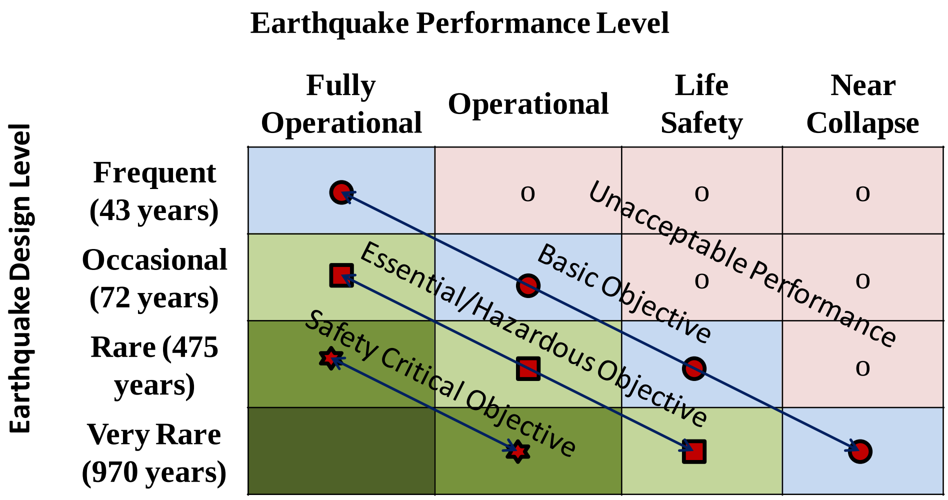

Figure 3: Recommended performance objectives for buildings in Vision 2000 (SEAOC 1995)

Performance-based methodologies were further modified in the following first generation PBEE documents (ATC 1996; FEMA 1997a; FEMA 1997b) that also used similar methodologies with slightly different performance and hazard levels. These documents incorporated seismic rehabilitation of existing structures and inspired a comprehensive PBEE guideline FEMA 356 (FEMA 2000). FEMA 356 qualitatively described different damage states for concrete frames as shown in Table 1. However, the absence of quantitative information of damage makes it rather difficult to assess the amount of repair activities and repair cost. In addition, a broader range of structural response parameters is required to properly quantify damage states.

Table 1: Criteria for Assigning Structural Performance Level to Concrete Frame Members, Reproduced from Table C1-3 in FEMA 356 (FEMA 2000)

Table C1-3 Structural Performance Levels and Damage—Vertical Elements				
Structural Performance Levels				
Elements	Type	Collapse Prevention	Life Safety	Immediate Occupancy
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent.	2% transient; 1% permanent.	1% transient; negligible permanent

The major limitations of first generation PBEE methodologies were those being deterministic in nature. Uncertainties in earthquake intensities, ground motion, structural response, damage and repair cost were not considered. To address these issues, the Pacific Earthquake Engineering Research (PEER) Center, one of the three earthquake engineering research centers in the US, has developed the PEER PBEE methodology -a probabilistic and comprehensive system-level performance assessment methodology. The

PEER methodology requires successive completion of four analysis stages: hazard, structural, damage, and loss analysis. A general overview of PBEE is available in Porter (2003), Deierlein et al. (2003) and Moehle and Deierlein (2004). Some of the recent developments in PEER PBEE can be found in Lee and Mosalam (2006), Goulet et al. (2007), Mitrani-Reiser et al. (2007), Haselton and Deierlein (2007) and Yang et al. (2009). These studies investigated the direct and indirect economic losses, downtime, number of fatalities and injuries for different benchmark buildings by systematically completing the four fundamental steps of PEER PBEE and properly accommodating uncertainty propagation within these steps. Recently, a new project known as ATC-58, sponsored by FEMA, has been completed to address the next generation PBEE design guidelines (ATC 2012).

2.2.3 Life-Cycle Cost Assessment Function of the Framework

LCCA is a decision-support or decision-making tool used for performance assessment in many engineering fields. It aims at determining the total cost of a facility, which comprises costs associated with different life-cycle stages. From a building's perspective, total or life-cycle cost can be defined as the sum of all expected costs from construction to the end of the structure's life span discounted to present value of money. LCCA is particularly useful for comparing different alternatives that meet the requirement in terms of performances but vary in terms of initial or total cost. Trade-off between performance and total life-cycle cost can be drawn in order to maximize net profits or savings. Therefore, LCCA is said to play an important role in design optimization by allowing future maintenance, repair and replacement costs to be included within the parametric study framework (Leeming 1993). For structures subjected to a hazard, life-cycle cost also refers to the future possible monetary losses due to unsatisfactory performance of the structure under forces with random occurrence and intensity during its life.

LCCA concept has been successfully implemented in various energy and water conservation projects, transportation projects (roads, pavements, highways, bridges) and building projects as a decision-support tool (Fuller and Petersen 1996). For building structures, the application ranges from different types of buildings (steel and reinforced concrete) under various extreme cases such as high corrosive environment and high seismic regions (Takahashi et al. 2004; Mitropoulou et al. 2011). In both cases, the inherent assumption is that the damaged or deteriorated structure is brought back to its original pre-hazard state, and repair/restoration cost of future hazards is incorporated within the life-cycle cost. The earlier application of LCCA in civil infrastructure mostly investigated the ownership and operating cost of a material, product, component, or facility over its service life without any consideration of unexpected future performance (Arditi and Messiha 1996; Asiedu and Gu 1998). Later, LCCA was implemented as a performance appraisal tool to ensure improved damage and cost reduction practices in extreme load cases.

A significant amount of research has been conducted from thereafter regarding estimating losses in terms of repair cost of structures subjected to earthquakes. Chang and Shinozuka (1996) proposed a conceptual framework that takes into account the potential discounted cost for seismic retrofit and damage repair in life-cycle cost estimation of highway bridges in high-seismic regions in addition to initial capital and discounted maintenance costs. Attempts were made to relate the framework with performance-based design codes. The framework was successfully applied to address several issues such as consideration of earthquakes in life-cycle cost formulation, selection of optimum maintenance, repair, and upgrade scheme, economic justification of seismic retrofitting procedures, and decision of a design performance level in various seismicity regions. Wen and Shinozuka (1998) investigated cost-effectiveness of control systems, which were used for minimizing seismic damage. Quantitative comparisons between controlled and uncontrolled structures were made through LCCA, and the use of control was justified for high excitation levels.

Wen and Kang (2001a; 2001b) proposed expected life-cycle cost functions for considering the cost of construction, maintenance and operation, repair, damage, and cost of failure consequence (loss of revenue, fatalities, and injuries, etc). Their proposed methodology was extended to incorporate LCCA of structures in multi-hazard phenomena. The method is applied to optimally design steel buildings subjected to both wind and earthquake hazards. In another study, Sarma and Adeli (1998) found a similar formulation to be impracticable due to scarcity of actual cost data. Takahashi et al. (2004) proposed a life-cycle cost formulation consisting of the initial cost and the expected damage cost due to future earthquakes. The design objective was to minimize the life-cycle cost for ensuring better seismic risk management. The LCC of each design alternative was evaluated using a renewal model. A Poisson model was adopted for characteristic and smaller earthquakes, respectively. The expected damage cost was obtained for earthquakes of specific magnitude for a fixed source, through simulation of source mechanism and ground motion characteristics, structural responses and damage cost generations. A case study was presented for a steel special moment resisting frames.

Kumar et al. (2009) presented a probabilistic approach to compute the life-cycle cost of RC bridges subjected to earthquakes and extended that methodology to consider the effects of aging due to corrosion. They found maintenance cost to be higher than the expected failure cost throughout the service life of the structure and suggested that maintenance and inspection might not be economically justified for non-critical structures. Mitropoulou et al. (2011) performed life-cycle cost assessment of an RC building, which was optimally designed taking into consideration the effect of seismic actions. Incremental static and dynamic analysis was used for assessing the seismic capacity of the building. They found the effect of interstory drift to be more vital than that of maximum floor acceleration in computing LCC. Symmetric structures were found to have lower LCC due to less susceptibility to damage compared to asymmetric structures. They addressed the importance of considering uncertainty in modeling, which is likely to influence LCC.

LCC has also been related with the structural responses of buildings. Kohno and Collins (2000) studied the variation of LCC with the change in base shear capacity. Nonlinear dynamic analyses were performed to obtain the structural response quantities. Structural damage costs were obtained from structural response metrics using the cost model adopted in HAZUS 99 (FEMA 1999) earthquake loss estimation methodology. It was observed that LCC results highly depend on assumptions regarding the cost model. Lagaros (2007) used LCCA for comparing the behavior of three vulnerable design practices for RC frames (soft ground storey, short columns, and their combinations) against earthquake hazards.

LCCA can also be used for managing and maintaining aging and deteriorating infrastructure. Frangopol et al. (1997) proposed a framework for reliability based life-cycle cost optimal design of deteriorating structures, where damage over time was modeled. The optimization methodology was developed to minimize the cost function while maintaining the reliability of the structure. A review of recent developments on life-cycle maintenance and management planning for deteriorating civil infrastructure can be found in Frangopol and Liu (2007).

Although mostly used as a decision support tool, LCCA has also been used for decision-making process, where future damage cost is used for obtaining cost-effective solutions through optimal design. An integrated framework considering lifetime seismic damage cost in the initial design phase was first introduced by Liu and Neghabat (1972). Thereafter, a significant number of studies have been performed regarding seismic design optimization using life-cycle cost as an objective function to be minimized. Sarma and Adeli (2002) developed a life-cycle cost optimization model based on fuzzy logic, which requires inputs from structural designers on relative importance of different design variables. While, Beck et al. (1999) implemented an economic performance parameter called probable frequent loss for assessing life-cycle cost. Probable frequent loss which was defined as the expected value of losses with a 10% POE in five years, and was found to be proportional to the expected seismic life-cycle cost. Unlike previous studies, Liu et al. (2004) considered initial material/construction cost and lifetime seismic damage cost as two separate measures. The lifetime seismic damage cost is computed in terms of POE of prescribed drift ratio limits that defines different damage states. Fragiadakis et al. (2006) incorporated life-cycle cost in multi-objective optimal design of steel structures to account for cost of expected damage from future earthquakes. The cost of expected damage includes cost of repair after a hazard, the cost of loss of contents, the cost of injury recovery or death, and other direct or indirect economic losses. The cost of exceedance of a damage state is obtained as a percentage of the initial cost. The uncertainties from ground motion parameters and seismic demand on the structure are integrated in the methodology. Kappos and Dimitrakopoulos (2008) determined the optimal retrofit level through cost-benefit and life-cycle cost analyses for RC buildings. A methodology for reducing seismic vulnerabilities due to retrofit was also proposed using fragility curves. More studies regarding life-cycle cost optimization is presented in section 2.3.

2.2.4 Life-Cycle Environmental Impact Assessment Function of the Framework

Traditionally, LCEIA has been defined as an integrated tool that provides quantifiable investigation and evaluation of the environmental impacts of a product, process or service associated with all the life cycle phases. The general methodology for LCEIA is based on the concept of environmental impact quantification proposed by the International Organization for Standardizations (ISO) in ISO 14040 series. ISO uses the generic term LCA instead of LCEIA, and the guidelines (for this) have been presented in two documents: ISO 14040 and ISO 14044 (ISO 2006a; ISO 2006b). Similar to LCCA, LCEIA considers the entire life-cycle of a product, process, or system encompassing the extraction and processing of raw materials; manufacturing, transportation and distribution, use, reuse, maintenance, recycling and final disposal. LCEIA has become a widely used methodology, because of its integrated way of treating the framework, impact assessment and data quality.

Several LCA studies were performed over the years to measure environmental impact using different boundary conditions, type of emissions, and environmental impact categories. A complete review of these studies can be found in Khasreen et al. (2009) and Sharma et al. (2011). Most of these studies concentrated on quantification and optimization of the non-structural aspect of the building life-cycle such as operational energy use and environmental emissions along with that of initial material production and construction stages. Environmental impact of structural repair activities was not directly considered in the operation/occupancy/use stage. However, environmental impact due to routine maintenance such as painting, polishing were employed for non-structural elements.

Adalberth et al. (2001) performed life-cycle environmental impact assessment to obtain the relative contribution of different life-cycle phases such as manufacturing, transportation, construction, occupation, and end-of-life (renovation, demolition and removal). Four different types of buildings were chosen in order to find out if there exists any relation between building type and environmental impact. Four environmental impact indicators viz. global warming potential (GWP), Acidification Potential (AP), Eutrophication Potential (EP), and human toxicity were considered. For both environmental impacts and energy use, occupational phase was the dominant phase, and good correlations were observed between environmental impact and energy. Since manufacture and construction were environmentally less significant, it was recommended to use energy efficient material to minimize the operational impact. Scheuer et al. (2003)

developed a complete environmental inventory of materials used for construction and replacement of structural components, envelope, interior, finishes, and utilities as well as end-of-life of a case study university building. Impact categories considered in the assessment comprised of primary energy consumption, GWP, ODP, AP, nitrification potential (NP), and solid waste generation. The whole life-cycle was divided into three phases: construction, operation and demolition. Operation, with 83% of the total environmental impact, appeared to be the most significant phase. Operational use of electricity and other energy sources for lighting and HVAC caused 94.4% of the total life-cycle energy consumption. All the studied impact categories were found to correlate well with the energy consumption.

Junnila and Horvath (2003) carried out a comprehensive LCEIA and data quality assessment of an office building in order to build a relationship between different elements or phases of life-cycle and potential environmental effects. The life-cycle of the building was divided into five stages: materials manufacturing, construction, use, maintenance, and demolition with transportation being incorporated in every stage. Climate change, acidification, dispersion of summer smog and heavy metals, and eutrophication were the studied impact categories. Electrical services used in lighting and HVAC, and manufacturing of concrete and steel were found to be primary contributors of environmental impact, whereas construction and demolition had relatively minor contributions. Other studies also had similar findings regarding relative contributions of different life-cycle phases. Sartori and Hestnes (2007) estimated that operation phase in conventional buildings represents approximately 80% to 90% of the life-cycle energy consumption and material extraction and production accounts for 10% to 20% of the same. Life-cycle energy consumption and CO_2 emission was studied by Suzuki and Oka (1998) for an office buildings in Japan. Norman et al. (2006) also used the same environmental indices in a comparative study of buildings using economic input-output (EIO) based life-cycle assessment. Some impact categories were omitted due to lack of reliable data.

Kofoworola and Gheewala (2008) also found operation phase to be the most dominant while performing LCEIA of a high-rise office building using process-based and EIO-based methodology. Both steel and concrete were found to be significant materials with respect to usage and environmental impact. Blengini (2009) performed LCEIA assessment on a residential building to be demolished in order to study the end-of-life phase using actual field measured data on the demolition of buildings and rubble recycling processes. Six environmental impact indicators i.e., gross energy requirement, GWP, ODP, AP, EP and Photochemical Ozone Creation Potential (POCP) were analyzed. The study showed that waste recycling is sustainable from economic, energy and environmental points of view.

Structural aspect of sustainability has been studied through comparative analysis between different construction materials and construction practices in terms of energy requirement and environmental emissions. In late 1990's some life-cycle inventories of environmental emissions were developed taking into account alternative construction materials, such as concrete and steel, and comparative LCEIA were performed to obtain the total environmental loads (Björklund et al. 1996; Jönsson et al. 1998). Individual building-level sustainability was sought in terms of energy consumption, harmful atmospheric emission, and depletion of natural resources by Johnson (2006). A detailed comparative study was performed with due consideration of all major product systems and material flows involved with construction of reinforced concrete and steel building in order to quantify the impact of building materials.

The environmental impact associated with construction phase was studied in detail by Guggemos and Horvath (2003) to develop an environmental input-output database associated with various construction units. Life cycle energy use and emissions of structural steel frames and cast-in-place concrete frames were evaluated. The contributions of construction, maintenance and end of life phases were negligible compared to the whole life-cycle. The construction phase of concrete frame used more energy and resulted in greater emissions of CO_2, CO, NO_2, particulate matter, SO_2, and hydrocarbon due to the use of more materials, heavier equipments and higher numbers of vehicles. On the other hand, it was found that the emission due to steel frame construction consisted of more volatile organic compound (VOC) and heavy metals (Cr, Ni, Mn) because of painting and welding of steel.

However, the above mentioned studies did not take into consideration the damage due to natural hazards, which is likely to have a significant effect on the total life-time environmental impact. Such an assessment requires evaluation of lifetime structural performance, as discussed in Section 2.2.2. Future structural damage due to natural hazards may require repair materials and equipment. Production and application of repair materials have environmental consequences such as resource depletion, GHG emission and energy use in the form of electricity or fuel. This in addition to the material lost due to premature failure of structural elements can be considered as service-life environmental loss.

There have been some recent studies on environmental loss assessment of structures under seismic hazards. Arroyo et al. (2012) introduced environmental loss consideration within seismic structural design methods. Environmental cost, defined as the product between the CO_2-equivalent emissions and the carbon tax, was found to be more than 1% of the cost of the facility. Designing the structure to withstand higher loads can sometimes reduce future environmental losses. Alternative materials can also significantly reduce CO_2-equivalent emission factors and environmental cost. Recently, Applied Technology Council (ATC) expanded their project for next generation performance-based seismic design guidelines to develop a performance based environmental impact assessment methodology. A methodology for quantification of environmental impacts due to seismic damage in terms of carbon footprint and other metrics was introduced in ATC 86. Potential environmental benefits, through reduction of life-cycle environmental impact, by employing performance based seismic design and retrofit has also been incorporated in the draft document (Court et al. 2012). Time-variant sustainability assessment of bridges was also performed under multi-hazards, i.e., simultaneous aging and deterioration, and the results were presented in terms of energy consumption and environmental emissions (Tapia and Padgett 2012; Dong et al. 2013).

2.3 Optimal Structural Design of RC Buildings

This section concerns the optimal design RC buildings considering the initial and life-cycle cost. Although material weight contributes to a major part of the total cost of a structure, weight optimization does not take into account other significant initial and life-cycle cost components such as labor cost, cost of formwork, repair cost, demolition and other end-of-life costs. For steel structure, construction cost is small compared to cost of steel production, hence the initial cost can be represented in terms of material weight only. Unlike steel structures, cost optimization is more appropriate for concrete structures as it involves more than one material. Hence, costs of concrete, reinforcing steel, labor and formwork need to be considered. Numerous studies have been performed on cost optimization of RC beams, columns, slabs and frames. These studies are grouped based on the number of objectives (single vs. multiple) and the optimization approach (mathematical programming-based, gradient-based or heuristic). After a brief summary of weight optimization studies, which mainly target steel structures, a detailed review of previous studies on cost optimization of RC buildings is provided.

2.3.1 Weight Optimization

Several studies in literature aimed at minimizing the weight of the structure based on the assumption that the cost is directly proportional to the weight (Feng et al. 1977; Cameron et al. 1992; Camp et al. 1998; Pezeshk 1998; Li et al. 1999; Memari and Madhkhan 1999; Foley and Schinler 2003; Lagaros et al. 2006; Liu et al. 2006, amongst others). Although this is, for the most part, true for steel structures, it is difficult to make such a correlation for RC structures. Therefore, studies on weight-optimal design of concrete structures are limited in comparison. These studies can be primarily divided into component-level and whole-structure optimization.

Weight optimal design of RC beam elements was performed by Chung and Sun (1994). The beam thickness and reinforcement area were considered as design variables with constraints on deflection, stress, and section sizes. Incremental finite element technique was used to unify structural weight optimization with structural analysis, design, and sensitivity analysis. Sequential linear programming algorithm was used to incorporate material nonlinearity in the formulation. Karihaloo and Kanagasundaram (1987) used linear and nonlinear programming techniques to solve weight minimization problem of statically indeterminate beams with constraints on normal and shear stresses. While, Karihaloo and Kanagasundaram (1989) proposed minimum-weight design of plane frames under multiple loads taking into account the effects of buckling and transverse deflections. Under some assumptions, the optimization problem was reduced to a non-linear programming problem and was solved using several methods: sequential convex programming, sequential linear programming, and sequential unconstrained minimization technique.

2.3.2 Single-Objective Cost Optimization

The objective function for the single-objective cost optimization problems is typically chosen as the initial cost of the structure comprising material and construction costs. Design variables comprise section sizes and reinforcement ratios for all the members. Various structural performance metrics as defined in the building codes are selected as constraints. Earlier attempts in structural optimization of building frames were more oriented towards the use of non-heuristic optimization techniques. An exhaustive review of literature on mathematical programming-based optimization can be found in Sarma and Adeli (1998).

2.3.2.1 Single-objective and mathematical programming-based optimization

Mathematical programming methods (or direct methods) are mostly linear and nonlinear programming techniques, which have been successfully applied to cost optimal design of RC structures. These methods were found to perform satisfactorily for limited number of design variables and constraints. Several notable studies used mathematical programming for cost optimization of RC structures (Hill 1966; Cohn 1972; Krishnamoorthy and Munro 1973; Cauvin 1979; Gerlein and Beaufait 1980; Kirsch 1983; Cohn and MacRae 1984; Huanchun and Zheng 1985; Krishnamoorthy and Rajeev 1989; Hoit 1991; Al-Gahtani et al. 1995).

2.3.2.2 Single-objective and gradient-based optimization

Mathematical programming optimization had less success in addressing feasible solutions for realistic optimization problems. On the contrary, gradient-based methods (or indirect) methods are found to be more efficient for large-scale optimization problems by taking into account numerous design variables and constraints. Use of gradient-based methods requires the existence of continuous derivatives of both the objective function and the constraints. For this reason, in most cases, analytical formulations are adopted to evaluate performance metrics. A review of selected studies on optimization of RC structures using gradient-based methods is presented in this sub-section.

Cheng and Truman (1985) developed a framework for optimal design of RC and steel structures using optimality criteria (OC) approach. Structural assessment was performed using elastic static and dynamic analysis. In order to meet the requirement of the used optimization algorithm, discrete member properties were converted to continuous variables. Structural weight (or cost) was chosen as the objective function subject to constraints on displacements. Moharrami and Grierson (1993) used OC method to determine the optimum cross-sectional dimensions and longitudinal reinforcement of the components of RC buildings subjected to constraints on strength and stiffness. Costs of concrete, steel and formwork form the objective function. Performance of the structure under gravity and static lateral loads was considered and evaluated based on prevailing code requirements. The results indicated that OC method converges smoothly to least-cost design and the final design is independent of the initial selection of the design variables.

Adamu and Karihaloo (1994; 1994) used discretized continuum-type optimality criteria (DCOC) for cost minimal design of RC beams with freely varying or uniform cross-sections along the span. Limiting values were applied on deflections, bending and shear strengths with bounds on design variables. The results were compared with that computed using continuum-type optimality criteria (COC) in another paper (Adamu et al. 1994). In a separate study the authors used the same criteria for RC frames with columns under uniaxial and biaxial bending actions (Adamu and Karihaloo 1995; Adamu and Karihaloo 1995). Design variables included width and depth of the members and reinforcing steel ratio. Deflection, bending and shear strengths were chosen as constraints. Fadaee and Grierson (1996) investigated the effects of combined axial load, biaxial moments and biaxial shear on three-dimensional RC elements. OC method was used for optimizing the sections sizes and reinforcement areas. Chan (2001) investigated optimal lateral stiffness design of tall RC and steel buildings using the OC method. The objective was to minimize the cost subject to lateral drift, stiffness and serviceability constraints. Constructability and practical sizing of members were also taken into consideration. The proposed method was applied to an 88-story building.

Chan and Zou (2004) utilized the principle of virtual work to generate elastic and inelastic drift response of RC building. Response spectrum and nonlinear pushover analyses were used respectively to produce those responses. The formulation was based on OC approach. A two-phase optimization approach was adopted. In the first phase, optimum member sizes were obtained through elastic design optimization. In the second phase, reinforcement ratios were found for previously determined sections through inelastic design optimization. Zou and Chan (2004), on the other hand, used OC method to minimize the construction cost of RC buildings subject to constraints on lateral drifts. Response spectrum and time history loading were applied based on Chinese seismic design code. Lateral drift response was formulated based on the principle of virtual work. Multiple earthquake loading conditions were taken into consideration for optimal sizing of members. Chan and Wang (2006) investigated the cost optimization of tall RC buildings subject to constraints on maximum lateral displacement and interstory drift. Member sizes were designed based on OC approach. Zou (2008) proposed an optimization technique for base-isolated RC buildings based on OC method. Similar to the author's previous study, lateral drift response was formulated based on the principle of virtual work. The underlying assumption of this study was that, all the members of the superstructure behave linear elastically while the isolation system behaves nonlinearly.

2.3.2.3 Single-objective and heuristic optimization

In spite of being computationally efficient, gradient-based approaches have limited scope since the objective function, constraints and their sensitivities are necessary. In addition, the search domain needs to be continuous, which prevents the use of discrete design variables such as the reinforcing steel areas or ratios. To circumvent these problems, researchers used the method of virtual work to explicitly define the objective function and constraints. The review in the previous section indicates that OC approach was selected as the gradient-based optimization algorithm in most of the cases. Recent advancement in computational tools, on the other hand, enables researchers to include computationally costly analysis methods, such as static pushover analysis and dynamic time history analysis in structural optimization problems, through finite element modeling. However, in most cases conventional gradient-based algorithms cannot be used because the continuity of functions or their derivatives may not exist. By using heuristic approaches, this problem can be overcome. Furthermore, heuristic algorithms can effectively find global minimum, while gradient-based algorithm might be trapped at a local minimum.

Genetic algorithm (GA) was first used as a technique to solve engineering optimization problem by Goldberg and Samtani (1987). Based on his study, many researchers successfully employed design optimization of structures. A comprehensive review of studies related to structural optimization based on GA is available in Pezeshk and Camp (2002). Choi and Kwak (1990) created a database of different RC sections sorted from the least to most resistance for obtaining optimum member design. A two-step algorithm, which involved finding the continuous and discontinuous solution from the database, was used. Design variables were reduced to a single one by using section identification numbers. Optimization of the entire structure was proposed by combining individually optimized elements. Similarly, Lee and Ahn (2003) developed a data set containing section properties of frame elements in a feasible range while performing discrete optimization of RC plane frames based on GA. The semi-infinite search space was converted to a finite one by using the data sets, which were later further modified and reduced based on the provisions of existing code regulations on reinforcement area and configuration. Camp et al. (2003) investigated material and construction cost minimization of RC frames based on GA. Serviceability and strength constraints were used to satisfy the code requirements and incorporated in the algorithm as penalty functions.

Balling and Yao (1997) used a multi-level approach for design optimization of RC concrete frames. RC frame optimization was identified to be more complicated than steel frames due to the complexities related to reinforcement design. The optimization of reinforcement detailing was simultaneously conducted with the optimization of cross-sectional dimensions. This approach enabled the investigation of the effect of reinforcement topology, bar selection, bar positioning, cutoff and bend points, and stirrups and ties. A simplified method, which is twice as fast as the traditional one, was also proposed based on the assumptions that either the lower bound of reinforcement area or strength would govern the optimum design. Similarly, Rajeev and Krishnamoorthy (1998) considered discrete design variables for detailing and placing of reinforcement in RC frames as opposed to traditional practice of selecting steel area as continuous design variables that required rounding up to realistic constructible values.

Govindaraj and Ramasamy (2005) studied the cost optimal design of continuous RC beams based on GA. Only the cross sectional dimensions of beams were considered as design variables in order to reduce computational costs. Constraints were applied on strength, serviceability, ductility, durability as per Indian standards. Detailing of reinforcement was accounted for in a sub-level optimization problem. Saini et al. (2007) performed cost-optimal design of singly and doubly reinforced concrete beams subjected to uniformly distributed and concentrated loads based on artificial neural networks (ANN). To bypass trapping of ANN in local minima, GA was used to optimize the architecture and user defined parameters. The limit state design and the optimization were performed with constraints on moment capacity, actual deflection and durability along with other geometric constrains according to Indian standards.

Sahab et al. (2005) proposed a two-stage hybrid optimization algorithm based on modified GA and applied this algorithm to perform cost optimization of RC flat slab buildings. In a similar study, Sahab et al. (2005) presented multi-level optimization procedure for RC flat slab building. Column layouts, along with section sizes and number of reinforcing bars were obtained through exhaustive search, whereas the hybrid optimization algorithm was used to find section sizes. Constraints were applied based on the design regulations. In a different study, in order to reduce the computational costs in finding optimal design of structures subjected to earthquake loads, Salajegheh et al. (2008) combined two artificial intelligence strategies: radial basis function (RBF) neural networks and binary particle swarm optimization (BPSO), and proposed a hybrid optimization method.

Leps and Sejnoha (2003) implemented augmented simulated annealing method for optimizing shape, bending and shear reinforcement of RC structures simultaneously While presenting an example for a continuous beam. Rao and Xiong (2005) proposed a new hybrid GA where GA was applied to determine the feasible search region that contains the global minimum. The optimum solution was obtained through an integrated algorithm comprising hybrid negative sub-gradient method and discrete one-dimensional search. An example was presented for optimal design of an RC beam. Ahmadi-Nedushan and Varaee (2011) applied Particle Swarm Optimization (PSO) method to one-way RC slabs with different support conditions. The total cost of the slab was selected as the objective function subject to constraints on strength, ductility and serviceability as recommended in the design code. A dynamic multi-stage penalty function was chosen which transforms the constrained problem to an unconstrained one by penalizing the impractical points on the search space. El Semelawy et al. (2012) found optimum values of slab thicknesses, number and sizes of tendons, and tendon profiles of pre-stressed concrete flat slabs based on modern heuristic optimization techniques. A general and flexible tool was developed that could handle real life problems. Costs of concrete and tendons were included in the objective function. Results suggested that the consideration of a second objective function (distance from constraints) would make the optimization technique more efficient.

Fragiadakis and Papadrakakis (2008) studied deterministic and reliability based optimization for designing RC frames against seismic forces and found the latter to be more feasible in terms of economy and flexibility of design. Non-linear response history analysis was performed for structural performance assessment. The objective was to obtain improved performance against earthquake hazards with minimal cost. Evolutionary algorithm (EA) was used to solve the optimization problem. Three hazard levels were considered. Several limit states from serviceability to collapse prevention were considered. Similar to what has been adopted in this study, to reduce the computational time, fiber-based beam-column elements were used only at the member ends, and inelastic dynamic analysis was performed only if non-seismic checks performed through a linear elastic analysis were met.

2.3.3 Multi-Objective Optimization

In most studies on single-objective optimization, the merit function was selected to minimize the cost of the structure through optimal material usage. Alternative designs were explored to obtain the optimal solution. Hence, single-objective optimization methods usually provide just one optimal solution. Decision makers either have to accept or reject the optimum design. On the other hand, multiple merit functions, which are related to decision making process, are taken into consideration in multi-objective optimization process. It offers decision makers the flexibility to select the "best" (or most suitable) option from a number of equivalent solutions based on their priorities and judgments. Hence, several studies formulated multi-objective optimization problem by modifying existing algorithms to account for multiple objective functions.

Ang and Lee (2001) formulated an integrated framework for structural optimization of RC buildings with respect to minimum life-cycle cost criteria and identified the fundamental safety and reliability of the building for each sets of design values. Life-cycle cost included initial costs from materials, labor, and construction together with probable damage cost from future earthquake hazards. By applying the minimum life-cycle cost criteria, constraints for the allowable risk of fatality were measured. Li and Cheng (2003) incorporated damage-reduction based structural optimization algorithm into seismic design of RC frames. Initial costs and total expected loss formed the objective function. A simplified approach for reliability analysis was adopted along with a tailored enumeration technique. Findings included improved seismic performance of damage-reduction-based design over traditional design, on the grounds of several metrics such as life-cycle cost, structural responses against extreme earthquakes and reliability of the weakest story based on the drift.

Lagaros and Papadrakakis (2007) compared two design approaches based on European seismic design code and performance-based design (PBD) for three-dimensional RC frames. The considered two objective functions were the initial construction cost and maximum inter-story drift. Linear and nonlinear static analyses were performed for European code based and PBSD, respectively. Three performance objectives corresponding to three hazard levels were considered. EA was used for optimization. Design based on Eurocode was found to be more vulnerable to future earthquakes. Zou et al. (2007) used OC method to minimize the initial material cost and life-cycle damage cost of RC frames in a multi-objective optimization framework for PBEE. Optimal member sizes were determined through elastic response spectrum analysis in the first stage of optimization. In the second stage, static pushover analysis was performed to find the reinforcement ratios. Fragiadakis and Lagaros (2011) presented an alternative framework for PBSD of structures adopting particle swarm optimization algorithm. The formulation could account for any type of analysis procedure (linear or nonlinear, static or dynamic). Initial cost or lifetime seismic loss could be selected individually or together to define the objectives of the problem. Both deterministic and probabilistic design procedures were incorporated. A number of limit states from serviceability to collapse prevention were selected for probabilistic design.

Paya et al. (2008) used cost, constructability, sustainability (environmental impact), and safety as the four objective functions while performing structural optimization of RC frames based on multi-objective simulated annealing (MOSA). Design was performed according to Spanish code. Pareto optimal set of solutions were obtained. Mitropoulou et al. (2011) used life-cycle cost assessment (LCCA) to evaluate the designs based on a prescriptive and performance-based methodology. Initial construction cost was minimized in the former case; while, in the latter case, life-cycle cost was considered as an additional objective function, turning the problem into a multi-objective one. Incremental dynamic analysis and nonlinear static pushover analysis were performed for structural assessment. Various sources of uncertainty were taken into consideration for seismic demand and structural capacity.

2.4 Conclusions from Literature Review

More than four decades of research is available regarding sustainability assessment and design optimization of civil engineering structures, and RC buildings in particular. Most of the studies concentrated on one or two aspects of sustainability namely, life-cycle environmental impact or life-cycle cost. Structural performance under hazard, which is essential for evaluating sustainability in high-seismic regions, was studied in very few of them. Since different functions of LCA are interrelated, it is essential to consider all of them to have a complete understating of the three sustainability components: total (lifetime) cost, environmental impact, and social impact.

Numerous studies exist with regards to structural optimization of RC buildings covering all aspects from component to system level, single to multi-objective, considering life-cycle cost, and taking into account seismic performances. However, to the knowledge of the author, optimization studies to minimize the impacts in all there sustainability components at the same time have not been reported. Therefore, the current study focuses at linking structural optimization with a comprehensive sustainability assessment framework in order to obtain a sustainable design optimized based on quantifiable metrics. A typical RC building subjected to high seismic loads was considered for this assessment.

3. A NEW LIFE-CYCLE ASSESSMENT FRAMEWORK FOR RC BUILDINGS

Although many researchers have investigated individual components of sustainability of civil infrastructure, comprehensive sustainability assessment efforts have been rare. Existing LCA studies have been mostly focused towards operational sustainability of building products not considering structural performance against earthquake hazards. Some loss assessment studies provide useful information regarding the importance of performance-based seismic assessment and outline the general methodology. However, these studies did not incorporate all the sustainability components in case of seismic events. Hence, there is a need for a comprehensive and well-defined framework for quantifying the sustainability of buildings covering all important aspects of sustainability that are directly associated with seismic hazards. This proposed methodology adopts PBEE technique and probabilistically assesses sustainability impacts by considering multiple levels of earthquake hazard using LCSPA function. LCCA and LCEIA functions convert probabilistic structural damage due to natural hazards to quantifiable metrics such as repair cost, environmental impacts, and downtime for a more reliable sustainability assessment. The methodology is extended to incorporate structural optimization, where the objectives are set to minimizing the economic, environmental and social impacts.

3.1 Sustainability Assessment Framework

A comprehensive framework for sustainability assessment of structures subjected to natural hazards is introduced herein. Economic and social impacts are included in sustainability assessment in addition to environmental impacts, which is the common measure of sustainability. Economic impacts are due to monetary expenses that are incurred during different phases of the structure's lifetime. Environmental impacts take into account environmental inputs in terms of natural resources and energy, and outputs in terms of environmental emissions and generation of wastes. Social impacts are relatively more difficult to determine since they involve dealing with socially sensitive parameters such as deaths. However, social impact also includes irrational components such as building aesthetics or inconvenience during construction which are subjective and qualitative in nature. It is more logical to consider social impact metrics that are quantitatively computable, and directly related to structural performance under seismic hazard. Downtime due to repair of damaged structure along with number of deaths and injuries after structural damage are reasonable metrics for social impact assessment.

In order to assess the seismic sustainability of RC buildings, the sustainability assessment framework outlined in Figure 4 is developed. The framework takes raw material, money and energy as the inputs of any project, and represents its influence on sustainability using total cost, environmental and social impacts. This framework has been developed to merge all the relevant life-cycle components through three interdependent functions: LCSPA, LCCA, and LCEIA. These case-specific functions convert the inputs (e.g., money, material) to obtain the output (e.g., emission), for all the relevant life-cycle stages. Subcomponents exist within the life-cycle stages and they vary depending on the type of structure. Inventories of resource, energy and monetary inputs are first developed followed by LCSPA, LCCA, and LCEIA considering relative inputs and outputs. Finally, environmental, social and economic impacts are assessed to assist the decision makers in trading off between alternative designs.

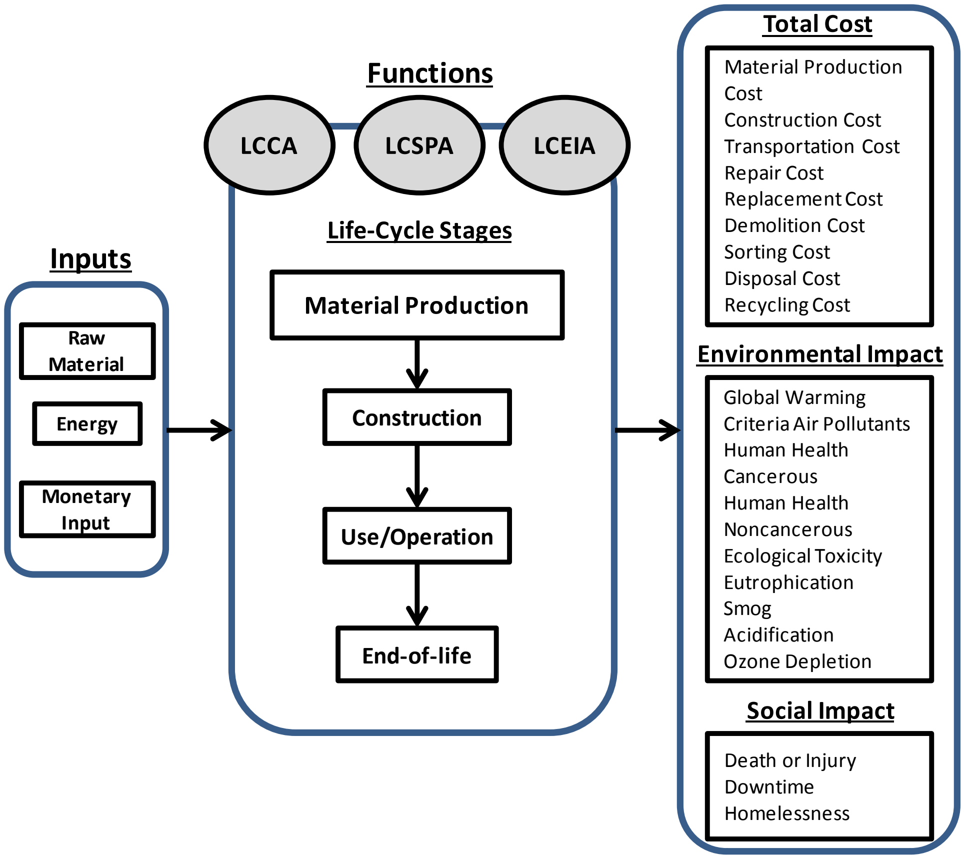

Figure 4: Life-cycle assessment framework for RC building (LCCA: Life-Cycle Cost Assessment, LCSPA: Life-Cycle Structural Performance Assessment, LCEIA: Life-Cycle Environmental Impact Assessment)

Every product exhibits various phases throughout its life. As a product, RC buildings undergo a number of life-cycle phases such as material production, construction, use, and end-of-life. Material production phase includes raw material extraction/mining, transportation of raw materials to the manufacturing plants, manufacturing of materials, and storage of finished materials. The construction phase consists of activities that take place during the construction process: transportation of finished materials and other products to the project site, on-site fabrication, use of equipment, and energy consumption of construction tools. The use phase includes all the activities that occur during the service life of the building including operation, maintenance, repair, replacement and retrofit. The end-of-life phase of a building takes into account the activities pertaining to demolition of the building, transportation of building debris to the sorting plant, sorting, transportation of sorted debris to recycling plant or disposal site, recycling of recyclable materials, and disposal of non-recyclable materials to landfill site.

A detailed flow diagram of life-cycle stages of typical RC buildings are shown in Figure 5. Extraction of raw materials is considered as the starting point, while the boundary ends with recycling of recyclable materials (e.g., steel and coarse aggregates) and disposal of remaining materials (e.g., fine debris). Since this study only focuses on the structural aspects of the building, the contribution of the nonstructural elements is not taken into consideration within the system boundary. Operational phase of LCA is studied in detail by green building practitioners who seek to optimize the life-cycle energy use and environmental emissions. Although this phase contributes to a significant portion of the total environmental impact, it is not included in the scope of this study because operational energy use and emissions during this stage are independent of structural design and performance under hazard. Even though the durability the structure is the key to long term performance, assessment of structures under combined effects of aging-induced deterioration and earthquakes is more involved and is also outside the scope of this study. Hence, time-variant deterioration and weakening of structure due to corrosion or sustained loads, which is likely to make the building more vulnerable against extreme hazards, are not taken into account during structural performance assessment.

Routine maintenance initiatives restore the structural capacity after these types of damage occur. Maintenance activities are also omitted from the framework because these are likely to remain constant for alternative designs, and are independent of seismic damage. As it can be seen in Figure 5, all four basic life-cycle stages: material production, construction, use, and end-of-life are included, while operation and maintenance sub-phases are kept out of the system boundary.

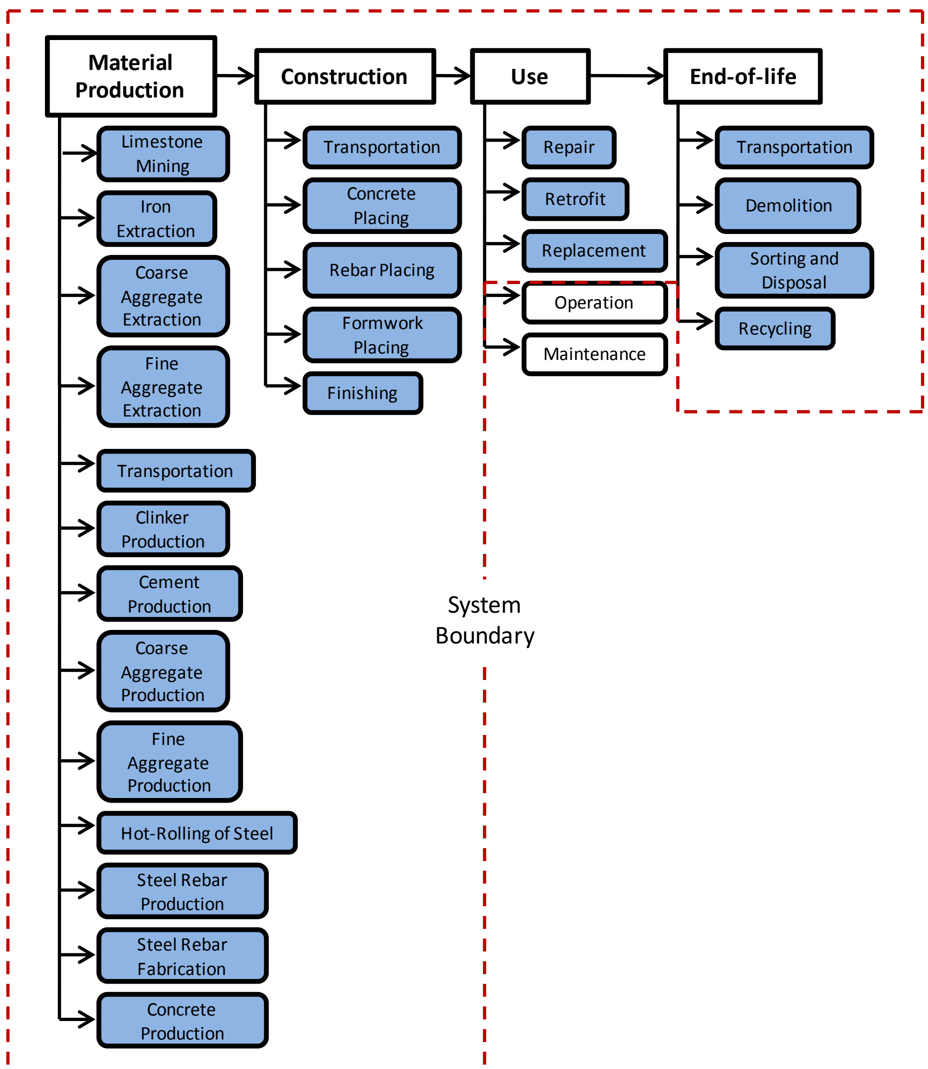

Figure 5: Life-cycle phases and selected system boundary

Additionally, the study investigates the impact of optimization in terms of total cost, total environmental impact, and total construction and repair time on the structural design of building. The optimal design reduces material usage by matching the capacity of the structure with the demand, and achieving a more reliable performance at all limit states. Initial design of buildings is accomplished with proper modeling and analysis. The optimal designs are obtained through the optimization module that minimizes the aforementioned components of sustainability considering the total service life of the structure. Comparisons between optimal and non-optimal design aid in decision-making process through trading off between structural performance parameters as well as different aspects of sustainability for individual design cases.

3.2 Life-Cycle Structural Performance Assessment

A general LCSPA methodology should investigate the effect of all factors that result in damage or deterioration of the structure that hampers the intended use of the facility. These sources include manmade and natural events with sudden or gradual consequences. As mentioned in the previous section, the focus of this study is placed on the structural damage incurred by RC buildings due to future earthquakes. However, the methodology proposed here can be extended to incorporate structural damage due to other hazards as well. For seismic structural assessment, earthquake hazard is properly modeled to accommodate earthquakes with different magnitudes and return periods. Structural performance under gravity loads is also evaluated for serviceability assessment of the building. Several performance metrics such as stress, strain, deformation, and length of plastic hinges at different critical locations of the structure, obtained using nonlinear structural analysis, are used to represent structural performance. These quantitative measures are eventually used to assess the probable damage state of the building, which is characterized locally by type and extent of cracking, spalling or crushing of concrete and yielding, buckling, or rupture of reinforcement, and globally by partial or complete collapse of the structure.

LCSPA is closely linked with both LCCA and LCEIA because the resilience of the structure directly affects both the life-cycle cost and the life-cycle environmental impact. Structural performance determines the amount of damage that the structure will experience due to a hazard. The probable damage of the structure obtained from LCSPA will eventually be decisive in the selection of a repair scheme and the associated repair cost and environmental load due to repair activity. Again, social impacts such as death and injuries, and downtime from repair activities are obtained through LCSPA, rendering LCSPA an essential component of the LCA framework

This section outlines a general procedure for LCSPA using the PBEE methodology presented by PEER Center (Deierlein et al. 2003; Porter 2003; Moehle and Deierlein 2004). The procedure presented here can be applied to all structures regardless of their type, location, use, age and occupancy. The main feature of this methodology is that the performance is represented in terms of various decision variables such as dollars, deaths, or downtime, which can be easily conceived by practicing engineers, policy makers and others users. In other words, engineering performance metrics are not directly used to represent the performance of a structure, rather these are translated to more easily understood measures such as repair cost, casualties, and downtime. Additionally, performance assessment is done probabilistically, taking into account uncertainties in all the steps of the procedure. A novel contribution of this study is that environmental impacts, in addition to commonly used economic and social indicators, are also assessed using the PBEE methodology. In PEER PBEE methodology, the performance assessment is divided into four successive steps: seismic hazard analysis, structural response evaluation, damage assessment, and loss analysis as shown in Figure 6.

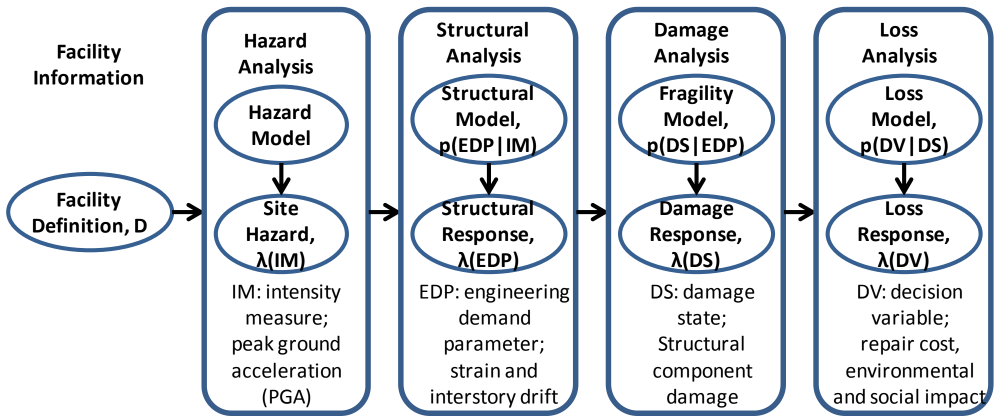

Figure 6: PBEE Methodology [adopted from Porter (2003)]

The reason behind adoption of PEER PBEE is that it is consistent with the proposed seismic sustainability assessment framework. The first three steps (hazard, structural, and damage analyses) constitute the LCSPA methodology, while loss assessment incorporates LCCA and LCEIA with proper selection of the loss function. In the following section, LCSPA methodology based on PEER PBEE is outlined along with the modifications incorporated in this study in order to relate structural performance with life-cycle sustainability.

3.2.1 Seismic Hazard Analysis

Seismic hazard analysis is the first step of LCSPA where earthquake hazards are taken into account probabilistically. Seismic effects primarily include direct and indirect consequences of ground shaking such as ground rupture, lateral spreading, liquefaction, landslides, and tsunamis among many others. However, the presented methodology only takes into account the effects of ground shaking on the facility response. Hazard analysis is performed accounting for the three primary ground motion parameters: magnitude, source-to-site distance, and soil conditions. These along with location of site, nearby faults and their mechanisms, recurrence interval of earthquake magnitudes are also studied. Ground motion attenuation relationships are used to incorporate these parameters in hazard analysis. Hazards are commonly presented in terms of one of these ground motion parameters commonly known as intensity measure (IM). Since hazard analysis is performed for a structure at a specific location, source-to-site distance and soil conditions are known parameters. Therefore, the peak ground acceleration (PGA) or spectral acceleration at the fundamental period of an equivalent single degree of freedom system, i.e., $S_a(T_1)$, are usually selected as IM. The outcome of hazard analysis is a hazard curve which demonstrates the relation between IM with its mean annual frequency of exceedence at a given site with due consideration of all the factors influencing ground shaking. The hazard curves obtained from probabilistic seismic hazard assessment assist selection of earthquake records at different hazard levels which is later used for structural analysis.

Hazard curve is developed by using the basic assumption that the occurrence of earthquake can be considered as a Poisson process (Kramer 1996). With this assumption, the hazard curve is drawn based on:

$$P(IM) = 1 - e^{-\lambda(IM)t}, \tag{1}$$

where IM is the selected intensity measure, $\lambda(IM)$is the annual frequency of exceedence, *t* is the service life of the structure, and *P(IM)* is the POE of IM in *t* years. Hazard curves are developed for various hazard levels defined by different return periods or POEs. In this study, three different levels of seismic hazard are defined with 75, 475 and 2475 years return periods (YRP) which correspond to 50, 10, and 2% probability of occurrence in 50 years, respectively. This selection was due to the fact that these hazard levels represent the three major structural limit states of performance based seismic design: immediate occupancy (IO), life safety (LS), and collapse prevention (CP), respectively. The relation between hazard levels and structural limit states are shown in Table 2.

Table 2: Relation between Hazard and Structural Performance Levels

Structural Limit State	**Return Period**	**Probability of Hazard**
Immediate Occupancy (IO)	75 years	50% in 50 years
Life Safety (LS)	475 years	10% in 50 years
Collapse Prevention (CP)	2475 years	2% in 50 years

The next step of hazard analysis is selection of earthquake ground motions compatible with each of the above mentioned hazard levels. A number of compatible ground motions that possess the property defined by the hazard curve are required. The number should be such that it would provide meaningful statistical data for structural analysis. Magnitude, distances, and site conditions relevant to the location of the structure are considered in conjunction with the IM while selecting the ground motions. Since it is rather difficult to find natural earthquake records that meet these criteria, the common way to obtain ground motions is scaling existing records in terms of IM values. Scaling records may sometimes produce ground motions that are not representative since scaling does not account for the change in duration, and it can amplify abnormal characteristics of the ground motion.

The PEER Next Generation Attenuation Relationship (NGA) database (PEER 2005) is used for ground motion selection for its completeness, reliability, and availability of a large dataset. Selected ground motions representing the hazard levels are properly modified in order to make it compatible with uniform hazard spectrum (UHS) for the given location. Spectrum matching is only needed for the critical period range of the structure taking into consideration the period elongation due to structural damage. A comprehensive methodology for ground motion selection and spectrum matching is available in Gencturk (2011).

3.2.2 Structural Response Analysis

In this step, the numerical model of the facility is developed and a suit of ground motions selected in hazard analysis step is used as inputs for nonlinear dynamic analysis. The objective is to determine the structural response in a probabilistic manner taking into consideration different hazard levels. Structural response is evaluated to obtain the engineering demand parameters (EDP) conditioned on earthquake hazards at different IMs. EDPs can serve as both component-level and system-level performance metrics for structural and non-structural components. Most common EDPs for global performance assessment are maximum interstory drift ratio and maximum floor acceleration for structural and non-structural components, respectively. However, other action and deformation based EDPs such as peak axial force, peak bending moment, peak shear force, maximum stress, maximum strain, peak plastic hinge rotation, and peak positive curvature are investigated mainly for local response assessment. Several damage or ductility-based indices can also be used as processed EPDs depending on the type of analysis and type of facility. The most notable of such indices are proposed by Park and Ang (1985), Powell and Allahabadi (1988), Fajfar (1992) and Mehanny and Deierlein (2000).

Structural performance assessment requires understanding and evaluation of both local and global behavior. Since non-structural issues are not within the scope of this study, only maximum interstory drift ratio for the whole frame is taken as the global EDP. In order to assess the global collapse state of the frame, maximum interstory drift ratio is compared with the collapse drift ratio, which is obtained through a prior pushover analysis. For a given structural frame, reverse triangular (code-based) load is applied at the end nodes of the frame, and the pushover curve (base shear vs. drift ratio) is developed. Collapse drift ratio is defined as the post-peak drift ratio corresponding to 10~15% reduction in the base shear value as recommended by Park (1988).

In addition to the global EDPs, maximum compressive and tensile strains of unconfined concrete (ε_{uc}), confined concrete (ε_{cc}) and reinforcing steel (ε_{s}) for each element along with the maximum interstory drift of each column are selected as local EDPs for all hazard levels. The strains are judged to be adequate for representing the structural responses of beams due to seismic load. However, maximum interstory drift of each column is included as an additional EDP for columns since it is a good indicator of component level response as well. Only the peak values of EDPs are required for any given analysis. Therefore, four EDP values for beams and five EDP values for columns are captured for each ground motion time-history. The novelty of this proposed selection of EDP is treating each structural component (beams and columns) of the frame individually, while previous studies (Mitrani-Reiser 2007; Yang et al. 2009) assembled relevant components of the buildings into different performance groups for simplicity and reducing the computational time.

Since nonlinear dynamic analysis is computationally expensive, it is rather difficult to obtain sufficiently large number of EDPs through analysis under a multitude of ground motions. On the contrary, having significantly large number of realizations is essential to properly account for uncertainty. In order to overcome this obstacle, additional correlated EDP generation technique proposed by Yang et al. (2009) is used to obtain an adequate number of EDPs. The procedure is based on the assumption that the EDP realizations of a given element exhibit jointly lognormal distribution. Generated EDPs are used for the following damage assessment step.

3.2.3 Damage Assessment

The third step of structural performance assessment is the damage analysis which involves translating EDPs to different damage categories that are easily comprehended by the policy-makers. The objective is to estimate the damage state (DS) which can be defined as the type and extent of damage of the structure at a component or at the system level. DSs can also be represented in terms of the effort required to restore the damaged components to their original pre-earthquake condition. The result of damage analysis is fragility curves which represent the probability that a component of the facility exceeds a particular DS conditioned on EDPs. Fragility relationships are obtained from experimental or numerical studies, expert opinions and/or engineering judgment. Some researchers congregate the components that show similar vulnerability in terms of the same EDP in damageable groups (Goulet et al. 2007; Mitrani-Reiser 2007; Yang et al. 2009; Ramirez et al. 2012). However, damage assessment of individual components rather than grouping similar components into damageable groups is also possible, and this approach provides more realistic representation of damage experienced by each element; therefore it is adopted herein.

EDPs from structural analysis of different components are used in the component fragility functions. The probability of having or exceeding DS 'j' for an element 'i' conditioned on IM is given as:

$$P\left[DM_{ij}\middle|im\right] = \int_{edp_i} P\left[DM_{ij}\middle|edp_i\right] p[edp_i|im]dedp_i, \tag{2}$$

where $P\left[DM_{ij}\middle|edp_i\right]$ is the POE the damage state 'j' for a given element conditioned on EDP 'i', and $p[edp_i|im]$ is the probability density of EDP 'i' for a given IM level.

Only damage associated with structural components is considered in this study. A single EDP (e.g., interstory drift) can be used to quantify all the structural damage states for a given element/damageable group. But in order to be more representative of the actual case, different threshold values are assigned for the selected EDPs in order to define different DSs. Each EDP is linked with a DS and associated repair effort.

Several DSs are defined in order to properly quantify and evaluate damage experienced by a structural element. In general, well-confined RC elements subjected to dynamic loads tend to follow the same progression of damage types: concrete cover cracking, yielding of longitudinal reinforcing bars, spalling of concrete cover, buckling of longitudinal bars, extensive damage in confined concrete followed by fracture of bars and crushing of concrete. However, buckling of longitudinal bar is excluded from the DSs due to the limitation of structural model in terms of capturing the initiation of buckling, and it is assumed to occur simultaneously with rebar rupture. Hence, seven DSs are defined in this study which is discussed in the following sections in detail.

DS_0: No damage or initial hairline cracking

Damage in RC components subjected to earthquake loading is typically initiated by cracking of concrete cover. As the crack propagates, the width and depth of the crack increases depending on various parameters. Crack width is typically defined taking into consideration the reduction of openings after the earthquake has ended commonly known as residual crack width. Hence residual crack width is used to determine appropriate repair type. However, getting an exact estimate of residual crack width is rather difficult. Hence, here maximum crack width is used for quantifying damage.

When the damage of concrete is limited to hairline cracks and the component does not experience any degradation in terms of strength and stiffness, the DS is defined as DS_0. The threshold crack opening for DS_0 is assigned to be equal to a maximum crack width of 0.8 mm (1/32 in). After structural analysis has been performed, crack width is calculated with reinforcing steel strain (ε_s) as the EDP from the equation provided by Frosch (2005) given as:

$$w_c = 2\varepsilon_s \beta \sqrt{{d_c}^2 + \left(\frac{s}{2}\right)^2}, \tag{3}$$

where,

w_c = maximum crack width, in

β = ratio of distance between neutral tension face to distance between neutral axis and centroid of reinforcing steel (taken approximately as $1.0 + 0.08\ d_c$)

d_c = thickness of cover from tension face to center of closest bar, in

s = bar spacing, in.

No structural repair efforts are required in this case. However, nonstructural repair such as application of surface finishes, paints, plasters, wallpapers, coats may be performed for aesthetic purposes and also to ensure water and fire resistance of the surface (Pagni and Lowes 2006). Since nonstructural repair is not considered in this framework, no repair initiatives will be undertaken if the component is in DS_0.

DS_1: Flexural cracking of concrete cover

If significant flexural cracking of concrete occurs, stiffness and strength degradation takes place. Therefore repair initiatives become necessary to restore the components to their original state. The structure lies in DS_1 if the crack width is between 0.8 mm (1/32 in) and 3.2 mm (1/8 in). The depth of cracks can be as high as 2.10 mm (1/12 in). If ε_s for a given column exceeds the threshold value given by Equation (3), the whole element is assumed to be in DS_1. Appropriate crack distribution pattern is assumed based on engineering judgment. The variation in crack width was not taken into account, and uniform crack width is assumed for the all the flexural cracks. Sufficient ductility is ensured by the structural design so that elements would fail due to flexure instead of shear.

Typically, flexural cracks are repaired manually (0.8 ~ 3.2 mm) or pneumatically (3.2 ~ 6.4 mm) by injecting epoxy resin or cementitious grout. The detailed procedure of structural epoxy injection is outlined in ACI RAP-1 (ACI 2003). The activities include cleaning/grinding of concrete surfaces, blowing out cracks with oil-free dry compressed air, installing surface-mounted entry ports, capping cracks at surface with epoxy gel, and finally grinding off epoxy cap residue after injection. Alternatively, holes are drilled along the cracks and epoxy is pushed to fill the holes. Sometimes, epoxy is injected on one side and vacuum is applied on the other side to pull epoxy and cover the whole crack region.

DS_2: Yielding of reinforcing steel

The next damage category DS_2 is defined by the initiation of yielding of rebars. A structural member is assigned DS_2 when reinforcing steel strain (ε_s) exceeds the yield strain (ε_y) of steel. Rebar yielding is usually repaired by applying RC jackets. The steps involved in jacketing are removing of damaged concrete, preparing interface surface, application of bonding agents, installing longitudinal and shear reinforcement, and application of concrete. Realistic assumptions are made while selecting the thickness and reinforcement ratios of the jacketed part for beams and columns. For example, the plastic hinge region is assumed to be equal to twice the depth of the beam and width of the column. The thickness of the jacket is taken as 152.4 mm (6 in).

DS_3: Spalling of concrete cover

Spalling of the concrete cover of RC elements is the next damage category. Spalling takes place when concrete cover loses bond between reinforcing steel and gets completely separated from the rest of the element. DS_3 is defined between the initiation of cover spalling and spalling of 30% of the surface area of the element. Unconfined concrete strain $(\varepsilon_{uc})_c$ is the governing EDP used for assessing the extent of spalling. Spalling is sometimes associated with buckling of rebars for flexural members. To restore the strength and stiffness, spalled surface of concrete is patched. First, the spalled and loosened concrete is removed, and the damaged region is thoroughly cleaned. Then mortar composed of sand, gravel and Portland cement is applied in the damaged area as per FEMA 308 guidelines (ATC 1998).

DS_4: Significant concrete damage

DS4 is defined by wide flexural cracks (greater than 6.35 mm), significant cover spalling (~50%), and crushing of confined concrete. The first two damage types are evaluated using the same procedure used for the previous DSs, whereas crushing of concrete is defined by the compressive strain of confined concrete $(\varepsilon_{cc})_c$. Crushing of concrete occurs when concrete placed inside the longitudinal steel concrete loses its load bearing capacity. In this DS, the extent of damage is such that cracks and spalled surfaces can no longer be repaired by epoxy injection and patching. As the rebars get exposed to the atmosphere, the bond between concrete and steel decreases significantly, and results in this type of damage. The strength and stiffness of crushed concrete cannot be restored without completely replacing the damaged portion. Hence, the repair measure taken for DS_4 is complete replacement of damaged concrete.

DS_5: Rebar rupture

DS_5 is characterized by rupture or buckling of reinforcing bars. In this damage state, RC components may experience complete failure. Ultimate fracture of rebar occurs when the maximum steel strain (ε_s) of a component exceeds the rupture or ultimate strain which is usually between 0.15 ~ 0.20. Since both lateral and vertical load carrying capacity might be lost, the whole component needs to be replaced in order to restore the strength and stiffness of the pristine element. The steps required include shoring the structure, removing concrete using chipping or jack-hammering, removing the damage sections of reinforcing steel, replacing the reinforcing steel, placing epoxy-embedded dowel bars as necessary, and replacing the concrete (Brown 2008). The steps are compatible with Structural Repair 4 as defined by FEMA 308 (ATC 1998).

DS_6: Global collapse

DS_6 is characterized by global collapse of the structure. It is defined in terms of maximum roof drift obtained from a prior pushover analysis. If the maximum interstory drift of any column exceeds the maximum roof drift, then the whole structure is assumed to collapse. The summary of all DS definitions are provided in Table 3.

Table 3: Definition of Damage States

DS	Damage Types	Damage Measure
DS_0	No damage or hairline crack	Crack width < 1/32”
DS_1	Open crack	1/32”<Crack width < 1/4”
DS_2	Rebar yielding	Initiation of yielding in rebar
DS_3	Moderate concrete spalling	30% surface spalling
DS_4	Wide crack Significant concrete spalling Concrete crushing	1/4”<Crack width 50% or more surface spalling Compressive failure of concrete
DS_5	Rebar rupture	$\varepsilon_s > 0.15$
DS_6	Collapse	Maximum interstory drift exceeding maximum roof drift

3.2.4 Loss Assessment

Loss assessment is the final step of the methodology that converts DSs to quantities that are useful for users, engineers and stake-holders for decision making based on the involved risk. This method probabilistically estimates decision variables conditioned on DSs of all components of the structure. Loss curves show the probability of exceeding a certain loss measure given that the facility is in a certain DS. Though PEER does not recommend any particular decision variable (DV) for loss assessment; various loss functions such as repair cost, repair duration, and number of casualties or injuries have been used by researchers over the years (Lee and Mosalam 2006). These three commonly known as death, dollar and downtime (three 'Ds') provide unique information for decision making under uncertainty.

In this study, damage and loss assessment methodologies are modified from the traditional approach. Rather than selecting one EDP for all the DSs of a structure, different EDPs are proposed in order for the assessment to be more representative. For example, cracking and spalling of concrete takes place because of tensile and compressive failure of unconfined concrete region, respectively, of a reinforced concrete member. Therefore selecting maximum tensile and compressive strains of unconfined concrete as EDPs gives a more practical representation for damage assessment. For loss assessment, various members of a structure is examined individually as opposed to the common practice of assuming that all the members of the same damageable group behaves similarly will undergo same amount of loss. The expected values of a particular DV can be presented as:

$$E(DV^n) = \sum_m \sum_i \sum_k E(DV^n | DS_k) p(DS_k | EDP_i{}^j)\, p\left(EDP_i{}^j \middle| IM_m\right) p(IM_m), \qquad (4)$$

where,

$p(IM_m)$ is the probability of the 'm'th value of IM

$p(EDP_i{}^j|IM_m)$ is the probability of the 'i' th EDP of the 'j' element for the 'm'th value of IM

$p(DS_k|EDP_i{}^j)$ is the probability of the 'k'th DS when subjected to the 'i'th value of the EDP of the 'j'th element

$E(DV^n|DS_k)$ is the expected value of the 'n'th DV for the 'j'th element of the facility when the 'k'th DS occurs

$E(DV^n)$ is the expected value of DV.

Loss Assessment of PEER PBEE methodology relates the proposed LCSPA with other life-cycle functions LCCA and LCEIA. The structural damage and the repair effort required for each component subjected to a suit of ground motions is achieved through LCSPA. Repair activities are then translated to associated cost, environmental impact, and time using the LCCA and LCEIA functions. Loss assessment using these functions is further discussed in the following sections.

3.3 Life-Cycle Cost and Environmental Impact Assessment Functions

The LCCA and LCEIA functions are described in this section using the LCA methodology proposed by ISO in ISO 14040 series (ISO 2006a; ISO 2006b) as a reference. The general LCA as outlined in ISO documents can be divided into four interacting stages: goal and scope definition, inventory analysis, impact assessment and interpretation as depicted in Figure 2 and discussed in this section.

LCA is performed to evaluate the economic and environmental loads at different life-cycle stages. The monetary input, raw materials and energy consumption, and emissions of by-products are listed for each activity through an input-output diagram. Later, the input-output flow of the whole product or process is obtained. These flows are represented in terms of functional units. Appropriate boundary conditions are used to define the scope of the study. For LCEIA, resource depletion or emission results are grouped based on their potential impact on the environment. Each emission result is weighted according to its contribution in the impact category in question. Finally, environmental impact of a particular stage or the whole product is obtained by adding the weighted values. Life-cycle environmental impact is represented by an environmental impact indicator through normalization of individual impact categories. The environmental impact indicator is then used to compare the environmental performance of alternative designs. In this study, several impact categories proposed by Environmental Protection Agency (EPA), Occupational Safety and Health Administration (OSHA), National Institutes of Health (NIH) and Building for Environmental and Economic Sustainability (BEES) are taken into consideration including but not limited to global warming, acidification, eutrophication, ozone depletion, eco-toxicity, fossil fuel depletion, smog formation, water use and human health. After calculating the potential raw material usage and environmental emissions from different life-cycle stages, the results are grouped and converted into these impact categories using Tool for the Reduction and Assessment of Chemical and Other Environmental Impact [TRACI] (Bare 2011), a tool proposed by EPA. However for LCCA, no such grouping and weighting is required since all impacts are presented in terms of monetary units. Detailed LCA methodology is presented in the following section. The interpretation step of LCA will be omitted in this section and described in Chapter 5.

3.3.1 Goal and Scope Definition

According to ISO 14040, goal and scope definition is the first phase of LCA, which involves identification of the product or process being analyzed and development of plans for conducting the assessment. The goal summarizes the reasons of conducting the study, the expected use and application of the outcome, and the prospective users and audiences of the study. The scope includes means to achieving the intended goals such as defining functional unit, system boundaries, and impact assessment methodology, among many others. Functional unit is the functional equivalence or quantitative reference of a product or process, which can be used as a basis for comparison with similar products or services. The functional unit has to be defined in detail in order to allow for comparison between equivalent designs and selection of the most optimal one. Defining a general functional unit for buildings is complicated. The functional unit of building is usually considered as the total usable floor area of the building or the amount of materials used to construct the building. The system boundary includes or excludes different life-cycle stages and their subunits depending on their relevance with the study as well as availability of data outcome. For example, if the objective of the study is to optimize construction practices, then the use or operation phase might be excluded from the system boundary. The data required for subsequent steps is also defined in goal and scope definition.

The goal in this LCA study is to incorporate the cost and environmental impacts resulting from seismic damage repair of the structural components of the building. Quantifiable sustainability parameters will assist selection of a more sustainable design. The results will inform building users, designers and policy-makers regarding the sustainability. The scope in this study is limited to the structural components of the total building (i.e., the functional unit). The system boundary (see Figure 5) is defined as cradle-to-grave with the exclusion of operation and maintenance, which are not directly related to structural performance.

Given the system boundary described in Section 3.1 above, the data required for conducting this study is the economic and environmental inputs and outputs corresponding to each of the life-cycle stages. For cost values, Building Construction Cost Data (RS Means, 2012) is a valuable resource covering in detail almost all aspects related to material production, construction and repair activities. However, for environmental impact, no single database exists that includes all the relevant information; therefore, several databases (EcoInvent, B-PATH) were merged here as described in Section 3.3.2.

3.3.2 Inventory Development for Environmental Impact Assessment

In this step, the inflow and outflow data corresponding to different processes of life-cycle stages within the system boundary are collected. Product flows from one unit of process to the subsequent unit are also considered during inventory development. Inventory of material manufacturing, construction processes, repair activities and end-of-life phases for a particular study are sometimes presented in terms of functional units different from the one defined in the goal and scope definition step. For example, the inventory for material production is usually represented in terms of total volume or weight of finished material. On the other hand, inventory for concrete crack repair is more related to the surface area of cracked region. Therefore, the inventories of individual processes need to be normalized and validated to the functional unit of the final product. The outcome of inventory analysis is life-cycle inventory (LCI) which lists the economic and environmental input-output flows of various life-cycle activities.

At first structural design is performed based on regulatory documents (building design codes) and the material quantities required to construct the building is assessed. Next, an inventory of all the inputs and outputs for each process are constructed. The cost and environmental impact inventory development is described in the following sections.

3.3.2.1 Inventory development for cost assessment

The cost inventory corresponding to material production, construction and end-of-life mostly depend on the type and amount of materials (e.g, concrete, reinforcing steel, formwork), their properties (e.g., ultimate strength, unit weight), structural design (e.g., section sizes, quantity and size of rebars), and architectural layout (e.g., topography, member length). Transportation cost for delivering materials and equipment from the shop to the site should also be accounted for in the the construction cost. Since these parameters are all related to the structural design of the building, cost associated with these life-cycle phases can be computed directly from the appropriate cost database.

Because of unavailability of data, some activities such as sorting and disposal that fall under end-of life stage is not considered. Use phase only incorporates structural damage repair cost. Cost of recycling and raw material extraction were included in the material production phase. Sometimes future death, injury due to unexpected performance and downtime for repair and restoration is converted to cost functions using the guidelines given by FEMA 227 (FEMA 1992) and ATC (1985). However, this study recommends incorporating these socially sensitive issues under social impact assessment category.

For inventory development, 2012 Building Construction Cost Data (RS Means, 2012) is used. The database results corresponding to the probable location of the structure (San Francisco Bay Area, California) is further analyzed in order to provide data in readily used format as shown in Table 4. In the table, the structural concrete is normal weight ready mix concrete composed of local aggregate, sand, and type I Portland cement with no admixtures. Reinforcing steel, both longitudinal and transverse, are detailed, bent, and cut at the shop. The total cost of these activities along with that of transporting steel from shop to site are included in the material cost. During database development, it is assumed that #7 to #11 sized rebars (English) are used as longitudinal bars, whereas #4 sized rebars are used as transverse steel. The cost of steel is based on a production of 50 to 60 tonnes. For construction cost of beams and columns, large beams and square columns with 609.6 mm (24 in) width are assumed. For demolition, the members are broken into small pieces weighing 5 to 10 tonnes, which are removed from the site later.

Table 4: Unit Material, Construction, Demolition Cost

Item	**Unit**	**Cost ($/Unit)**
Material Cost		
Concrete (35 MPa)	m^3	161.34
Longitudinal steel (A615 grade 40/60)	Ton	1097.78
Transverse steel (A615 grade 40/60)	Ton	1129.8

Item	Unit	Cost ($/Unit)
Formwork	m^2	35.09
Construction Cost		
Placing concrete in beam	m^3	56.52
Placing concrete in column	m^3	55.41
Placing longitudinal steel in beam	Ton	699.15
Placing transverse steel in beam	Ton	823.58
Placing longitudinal steel in column	Ton	1179.08
Placing transverse steel in column	Ton	1244.25
Placing formwork	m^2	132.5
Demolition Cost		
Breaking into small pieces	m^3	92.21
Removal of pieces	Ton	20.66

For calculating the total repair cost of a facility, unit repair cost is needed for each damage state defined in Section 3.2.3. RS Means is also used as the source of repair cost data. For DS_1, no repair is required; hence the unit repair cost is zero. Although the crack width may vary between 0.80 mm (1/32 in) to 6.35 mm (1/4 in), average repair cost value corresponding to flexural cracks with 3.18 mm (1/8 in) width and 254 mm (10 in) depth is used for all elements falling under DS_1. For concrete jacketing, unit cost includes cost of production and construction assuming 4% reinforcement ratio in the jacketed region. For DS_4 and DS_5 damaged concrete and the whole element are replaced, respectively. The cost includes material and construction cost, which also accounts for removal of the damaged part. Table 5 outlines the processed data from RS Means for repair of RC elements.

Table 5: Unit Repair Cost

DS	Repair Method	Units	Cost ($/unit)
DS_0	No repair	-	0
DS_1	Epoxy Injection to open cracks	m of crack length	45.28
DS_2	Concrete jacketing	m^3 of jacket	1388.71
DS_3	Patching of spalled surface	m^2 of spalled surface	339.92
DS_4	Complete replacement of damaged concrete	m^3 of damaged concrete	653.32
DS_5	Element replacement	m^3 of concrete and Ton of steel	653. 32 for concrete 2342.03 for steel
DS_6	Reconstruction	Total frame	Initial cost

3.3.2.2 Environmental input-output inventory development

Environmental emission database from different sources are integrated in order to develop a complete inventory. For material manufacture and construction, B-PATH spreadsheets developed by Lawrence Berkeley National Laboratory [LBNL] (Stadel et al. 2012) are used. This database provides complete LCI for production of concrete and steel. LCI of certain on-site construction activities such as placing of concrete, forming and fabrication of rebar, and fabrication of formwork are obtained from Guggemos (2003), which is also used in B-Path models. The inventory for the end-of-life phase consisting of demolition, sorting and disposal, and recycling activities is obtained from EcoInvent v2.0 (Frischknecht et al. 2007) database. Transportation distances between different units of production, construction and end-of-life phases were decided based on realistic assumptions.

Recycled steel data is used for reinforcing bars assuming 70% of raw materials used for rebar fabrication come from recovered scrap after a structure is demolished in the end-of-life phase. The recovery and recycling process produces environmental emissions. On the other hand, material recovery reduces consumption of virgin raw materials which results in a positive environmental performance. Since the database incorporates the environmental emissions and resource recovery in the material production phase of steel, recycling of steel is not separately studied in the end-of-life phase. It is assumed that after demolition, building debris is transported to the sorting plant which is the usual case for RC buildings. Steel and coarse particles are sent for recycling and fine particles are deposited in landfills. The Emissions and Generation Resource Integrated Database [eGRID] (EPA 2006) provided by EPA is used for obtaining emissions due to electricity use.

3.3.3 Life-Cycle Cost Assessment

The general cost function for a RC building subjected to seismic hazard can be written as:

$$C_{Total} = C_P + C_C + C_R + C_{E}, \tag{5}$$

where C_{Total} is the total cost, C_P is the material production cost, C_C is the on-site construction cost, C_R is the repair cost due to future hazard, and C_E is the end-of-life cost. In the cost function, C_P, C_C and C_E are considered as fixed costs because they solely depend on the initial design of the structure, and not on the structural damage due to future earthquakes. Therefore, these can be directly calculated deterministically from the initial structural design.

On the other hand, repair cost assessment requires the cost inventory to be integrated with the loss assessment step of LCSPA explained in Section 3.2.4. The cost values associated with each DS, provided in Table 5 is assumed to be deterministic. Hence placing unit cost values from Table 5 to Equation (4), gives the loss curve with respect to cost. The area under the loss curve provides the annual repair cost over the service life of the structure. Multiplying the annual repair cost with the expected service life of the structure gives the total repair cost, C_R.

It should be noted that discount rate is not considered in the cost function. The purpose for using discount rate is to convert future cost to net present value in order to allow comparisons. A reasonable value for discount rate is usually taken between 3-6%. The reason for not considering discount rate is due to fact that the cost database used for assessing future costs for repair and demolition is based on present market value. These values are likely to increase in the future based on historical cost index, which is around 2-3%. Therefore, it is assumed that the effect of discount rate on future costs will be compensated by historical cost index. Thus additional computation for addressing discount rate is not required.

3.3.6 Life-Cycle Environmental Impact Assessment

In the impact assessment step, the inventory outputs are processed to translate the inflows and outflows in terms of environmental impacts. These environmental impacts are introduced to evaluate the potential hazardous effects of emissions and resource consumption on the eco-system and human health globally or locally. Environmental by-products that contribute to the same environmental impact category are grouped together in this step. Environmental impact categories are listed in Table A1 along with the corresponding emission LCI data. For example, greenhouse gases (GHG) such as carbon dioxide and methane cause global warming, hence they are classified under global warming potential (GWP) impact category. Again, each inventory flow has different relative contribution on the impact. Therefore, the inventory flows are assigned weighting factors. For example methane has 23 times more GWP than CO_2.

Environmental impact can be represented in terms of a single index which takes into account the effect of all impact categories. Since different impact categories are expressed in terms of various emission equivalents, the impacts need to be normalized and weighted before they can be translated into a performance index. Normalization data for impact categories in line with TRACI has been proposed by US EPA (Bare 2011). Normalization factors correspond to environmental impact due to environmental flows in the US per year per capita. The normalized impacts are comparable with each other since they are dimensionless and have a common basis. Building for Environmental and Economic Sustainability (BEES) has also recommended converting normalized impacts to an Environmental Performance Score [EPS] (Lippiatt 2007). In order to obtain EPS, weighting factors are selected based on the relative long- and short-term effects on the environment. Here, the purpose of using a single index is to simplify the LCEIA output, to make equivalent designs easily comparable, and thus aid the decision making process. Some of the impact categories such as fossil fuel depletion and water intake are not available in TRACI; therefore, the weighting factors recommended by BEES are modified as shown in Table 6.

Table 6: Normalization Values and Weighting Factors (Normalization Values are per Year per Capita unless Indicated Otherwise)

Impact Category	**Normalization Value**	**Weighting Factors**	
		BEES	**This Study**
Global Warming	25 582 640.09 g CO2 equivalents	29.3	39.91826
Fossil Fuel Depletion	35 309.00 MJ surplus energy	9.7	-
Criteria Air Pollutants	19 200.00 microDALYs	8.9	12.12534
Water Intake	529 957.75 liters of water	7.8	-
Human Health Cancerous	158 768 677.00 g C7H7 equivalents	7.6	10.35422
Human Health Noncancerous		5.3	7.220708
Ecological Toxicity	81 646.72 g 2,4-D equivalents	7.5	10.21798
Eutrophication	19 214.20 g N equivalents	6.2	8.446866
Habitat Alteration	0.00335 T&E count/acre/capita	6.1	-
Smog	151 500.03 g NOX equivalents/year/capita	3.5	4.768392
Indoor Air Quality	35 108.09 g TVOCs	3.3	-
Acidification	7 800 200 000.00 millimoles H+ equivalents	3.0	4.087193
Ozone Depletion	340.19 g CFC-11 equivalents	2.1	2.861035

As described above, using the environmental inventory, these interactions are presented in terms of emissions and resource consumptions, and finally translated to environmental impact. For material production, construction, and end-of-life phase, these steps are directly related to the amount of material used for construction, which is governed by the initial structural design. Fuel consumption for transportation and construction, which are subunits of these life-cycle stages, is also a function of the structural design because higher material demand leads to more trips and more usage of machineries. Using the above mentioned databases, different impact categories are directly represented in terms of to the amount of materials used in each life-cycle phase. A sample dataset for San Francisco Bay Area, California is shown in Table 7. For a reinforcement ratio value between 1 to 8% (which is the code recommended limit), it was found that the amount of concrete usually governs the environmental impact during construction phase. Wherever variation of environmental impact is observed corresponding to different reinforcement ratio, conservative values are used for developing the dataset.

The environmental impact corresponding to the initial and end-of-life phases can be easily calculated from structural design variables and the unit impact values given in Table 7. For repair phase, the amount of repair materials required corresponding to each DS is first evaluated. Then using the loss function provided in Equation (4), the repair environmental impact is found. The total environmental impact is then obtained by summing up the values corresponding to each life-cycle phases.

Table 7: Environmental Impact Corresponding to Initial and End-of-Life Phases

Impact category	**Unit**	**Environmental Impact, per tonne**				
		Concrete Production	**Steel Production**	**Construction**	**Concrete Demolition**	**Steel Demolition**
Acidification (air)	H+ Moles eq	31.787	430.1	40.896	1.8583	26.5910
Ecotoxicity (air)	kg 2,4-D eq	0.3721	4.2033	0.0082	0.0004	0.0054
Ecotoxicity (water)	kg 2,4-D eq	0.0406	0.3224	0.0001	0.0001	0.0013
Eutrophication (air)	kg N eq	0.0213	0.1392	0.0353	0.0020	0.0286
Eutrophication (water)	kg N eq	0.0013	0.0196	0	0.0000	0.0000

Impact category	Unit	Environmental Impact, per tonne				
		Concrete Production	**Steel Production**	**Construction**	**Concrete Demolition**	**Steel Demolition**
Global Warming (air)	kg CO2e	135.8	1122.5	89.965	3.2330	46.2634
Human Health Cancerous (air)	kg benzen eq	7.9845	110.68	0.018	0.0001	0.0017
Human Health Cancerous (water)	kg benzen eq	0.6936	4.3678	0	0.0001	0.0009
Human Health Noncancerous (air)	kg toluen eq	1608.9	11293	2.5205	0.2407	3.4438
Human Health Noncancerous (water)	kg toluen eq	1048	6234.8	0.0773	0.1123	1.6064
Human Health Criteria	kg PM2.5 eq	0.0045	0.7018	0.0067	0.0001	0.0016
Ozone Depletion (air)	kg CFC-11 eq	0.000002	0.0035	0	0	0
Photochemical Smog (air)	kg Nox eq	0.585	4.5826	0.9902	0.0612	0.8755

3.4 Social Impact Assessment

Social Impact Assessment can be defined as a methodology to investigate the social effects of buildings resulting from construction, use, and other interventions such as demolition and repair. Therefore, Social Impact Assessment of building provides information regarding the intentional or unintentional consequences on the society, which can be positive or negative, from different life-cycle stages of the building. It is a rather ambitious task to integrate all social impact factors and represent them in terms of a social performance indicator. The primary reason is that most social impacts are subjective and unquantifiable. For example, Sakai (2013) proposes building aesthetics such as visual appearance to be included in social impact, which cannot be evaluated quantitatively and may not be important with regards to the function of a particular structure. Therefore, social impact parameters should be selected based on the design objective, functionality, and purpose of the structure. Since the focus in this study is seismic performance assessment, it is important that social indicators to be assessed can somehow reflect the building performance against hazard.

Over the years, unsatisfactory building performance against earthquake has led to a tremendous life loss. Also, bringing the structures in their original pre-hazard condition takes a substantial amount of time. These two indicators are direct functions of building's resilience and can be easily quantified in terms of numbers. Hence, fatality and downtime are recommended here as measures of social impact in the sustainability framework. However, fatality and injury estimations require reliable data about the spatial and temporal distribution of occupants along with much detailed damage and collapse quantification techniques which are not possible within the scope of the study. Therefore, no specific methodology is developed for the estimation in herein. For general overview of fatality and injury estimation, the readers are referred to Liel (2008) and Mitrani-Raiser (2007).

Downtime is traditionally known as the time required to repair the damage sustained by a structure. Historically repair time is found to be only a component of downtime for seismic events. Therefore, it is more justifiable to define downtime as the period between the closing down of a facility after the occurrence of a seismic event and the completion of building repair efforts that make it operational again. The other component of downtime is the time lag between the event and the beginning of repair incentives commonly known as the mobilization time. While the repair time is largely dependent on the damage state and repair scheme, mobilization time varies with case specific activities. These include repair inspection and re-inspection, consultations with professional engineers, and the contractor bidding process, financing, relocation of functions, human resources, and economic and regulatory uncertainty. The major issue with the assessment of the mobilization time is uncertainties associated with availability of labors, materials, and monetary input (Krawinkler and Miranda 2004). Since the mobilization time is mostly scenario specific, it is a demanding task to develop a generalize methodology that relates it with the initial design, structural response, and damage state. Hence, the methodology proposed herein only incorporates the rational part of downtime, which is the time needed to repair damaged buildings, and will be referred to as repair time henceforth.

The repair duration for each component, having different damage states, is related to the number and quality of repair crew. Therefore, it is more logical to represent repair efforts in terms of labor-hour, which generally means the amount of work produced by a skilled worker in one hour. Afterwards, the total work hours required for one person to complete the construction or repair activities is calculated. The total repair time is also calculated based on the damage states of individual components. RS Means database provides some repair time data (epoxy injection and patching), while the others (jacketing and replacement) are found from engineering judgment. For the latter, the amount of materials required for repair is first evaluated. Then, the repair time is assumed to be the same for constructing an element containing the calculated amount of materials. Table 8 provides unit repair time for RC elements associated with different damage states. The repair time is calculated from loss curves, similar to cost and environmental impact.

Table 8: Unit Repair Time

DS	Repair Method	Units	Repair Time (Man-hour/unit)
DS_0	No repair	-	0
DS_1	Epoxy Injection to open cracks	m of crack length	0.60
DS_2	Concrete jacketing	m^3 of jacket	7.62
DS_3	Patching of spalled surface	m^2 of spalled surface	3.33
DS_4	Complete replacement of damaged concrete	m^3 of damaged concrete	0.93
DS_5	Element replacement	m^3 of concrete and Ton of steel	0.93 for concrete 6.70 for steel
DS_6	Reconstruction	Total frame	Initial construction time

3.5 Multi-Objective Structural Optimization

Cost reduction of the structure has always been considered as one of the primary objectives in engineering design along with safety and serviceability. Due to the gradual diminishing of natural resources and substantial increase in construction materials and labor, the significance of structural design optimization has increased in the last decades. In addition to that, sustainable development has also promoted the practice of optimal material usage. The structural components of a residential of office building costs relatively less compared to nonstructural features such as partitions, floor and wall finish, mechanical, electrical, plumbing features. Nevertheless, reducing the initial structural cost is important both from financial and sustainability standpoints. Again, the total initial cost is found to be only 10 to 20% of the life-cycle cost for a typical reinforced concrete building. For high seismic region, unexpected structural performance can increase the total cost by a considerable amount. Therefore, from sustainability and seismic aspects, total life-cycle cost is a better substitute as an objective function than the initial cost. Considering other components of sustainability as objectives is necessary for finding an optimal sustainable solution in order to ensure sustainable development during the design phase of building.

3.5.1 Problem Formulation

Multi-objective optimization problem offers flexibility to the designers in selecting the most suitable design from a number of equivalent solutions. It overcomes the limitations of having a single merit function over which the decision makers do not have any control. Hence, it is a common approach to incorporate performance metrics in addition to the cost. The performance metrics provide useful information about the response of the structure with a particular design under seismic loads. However, the response parameter does not provide any direct information about the sustainability of the design in quantifiable terms. For this reason, this study proposes inclusion of all sustainability components as objective functions in order to allow selection of the most sustainable design. Hence, the objectives of the optimization problems are selected as the total life-cycle cost as the metric for economy, the total environmental performance score as the environmental impact metric, and the total construction and repair time as the social impact metric. The selection of sustainability metrics to be minimized is in line with the sustainability assessment framework as depicted in Figure 4. Proper integration of these metrics in the optimization formulation results in one or more optimally sustainable design solutions. The optimization problem is formulated subject to strength and serviceability constraints as per design code provisions (ACI 2008; ICC 2009) given by

$$\text{Minimize: } [C_{Total}, EI_{Total}, SI_{Total}]. \tag{6}$$

3.5.2 Seismic Structural Optimization Methodology

Design optimization of RC structures is a computationally demanding task, especially when inelastic dynamic analysis is used to evaluate structural response. The life-cycle assessment proposed uses a suit of ground motions corresponding to different hazard level. According to the regulatory documents, a minimum number of ground motions are required corresponding to each hazard level. This means that calculating the total cost, total environmental impact and social impact for a given design requires performing several nonlinear response-history analyses. However, all pre-selected design combinations may not conform to code requirements. Computational cost can be significantly reduced if these design combinations are eliminated while performing dynamic analysis. To partially overcome the problem with high computational demand, here a two level approach is adopted. The flowchart for two-level seismic optimization procedure is provided in Figure 7.

First, an elastic static analysis is performed for a selection of design variables. All the design checks, as mentioned in different regulatory documents such as ACI 318-08 (ACI 2008) and IBC 2009 (ICC 2009) are performed. If the design is classified as acceptable, inelastic dynamic time history analyses are performed under the selected ground motions. The EDPs are obtained from these analysis are stored and later further processed to calculate the total cost, total environmental and social impact. On the other hand, if the design is classified as unacceptable, the sustainability objective functions are assigned a large value so that this combination of design variables is penalized and not further considered by the optimization algorithm. All objectives are only evaluated if the selection of design variables is acceptable. For details of the total cost, total environmental impact and social impact calculations under seismic actions, see Sections 3.2 through 3.4.

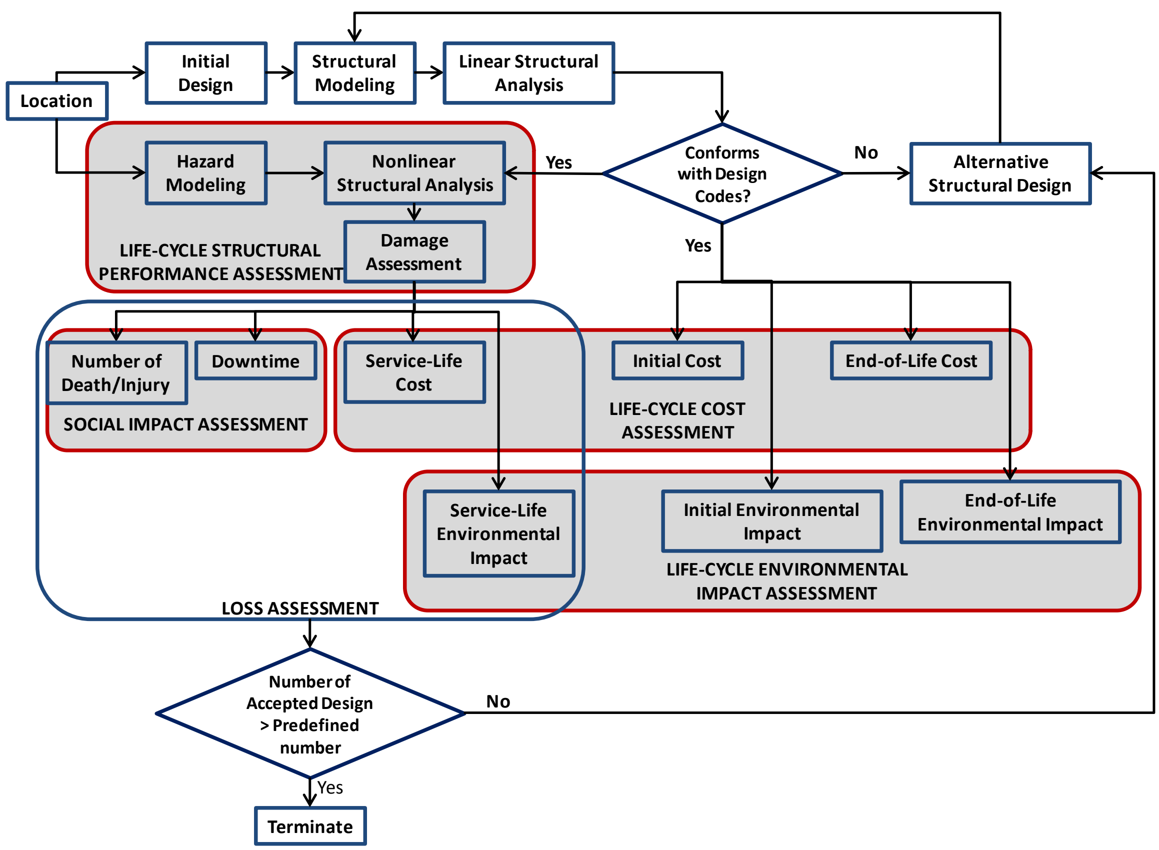

Figure 7: Seismic structural optimization flowchart

3.5.3 Tabu Search Algorithm

Although gradient-based techniques are computationally efficient compared to mathematical programming methods for large-scale optimization problems, they are limited to problems that have continuous derivatives of both the merit function and constraints. The sustainability metrics and the structural response parameters of an RC frame under earthquake excitation are highly nonlinear. Therefore, there is a possibility that the solution might get trapper at a local minimum. Additionally, gradient-based methods are not suitable for discrete design variables (e.g., cross-sectional area of elements and reinforcement ratio) that are commonly used for RC structures. Due to these reasons, Tabu search (TS) algorithm is used to obtain the optimal solutions for the multi-objective optimization problem considered here.

TS algorithm has been applied to various structural optimization problems and it has been showed to be very effective in solving combinatorial optimization problems with nonlinear objective functions and discontinuous derivatives (Bland 1998; Manoharan and Shanmuganathan 1999; Ohsaki et al. 2007; Gencturk 2012). TS employs a neighborhood search technique that sequentially moves from a combination of design variables **x** (e.g., section sizes and reinforcement ratios) that has a unique solution **y** (e.g., total cost, environmental and social impact), to another in the neighborhood of **y** until some termination criterion is reached. Details of the TS algorithm used here are given in Figure 8.

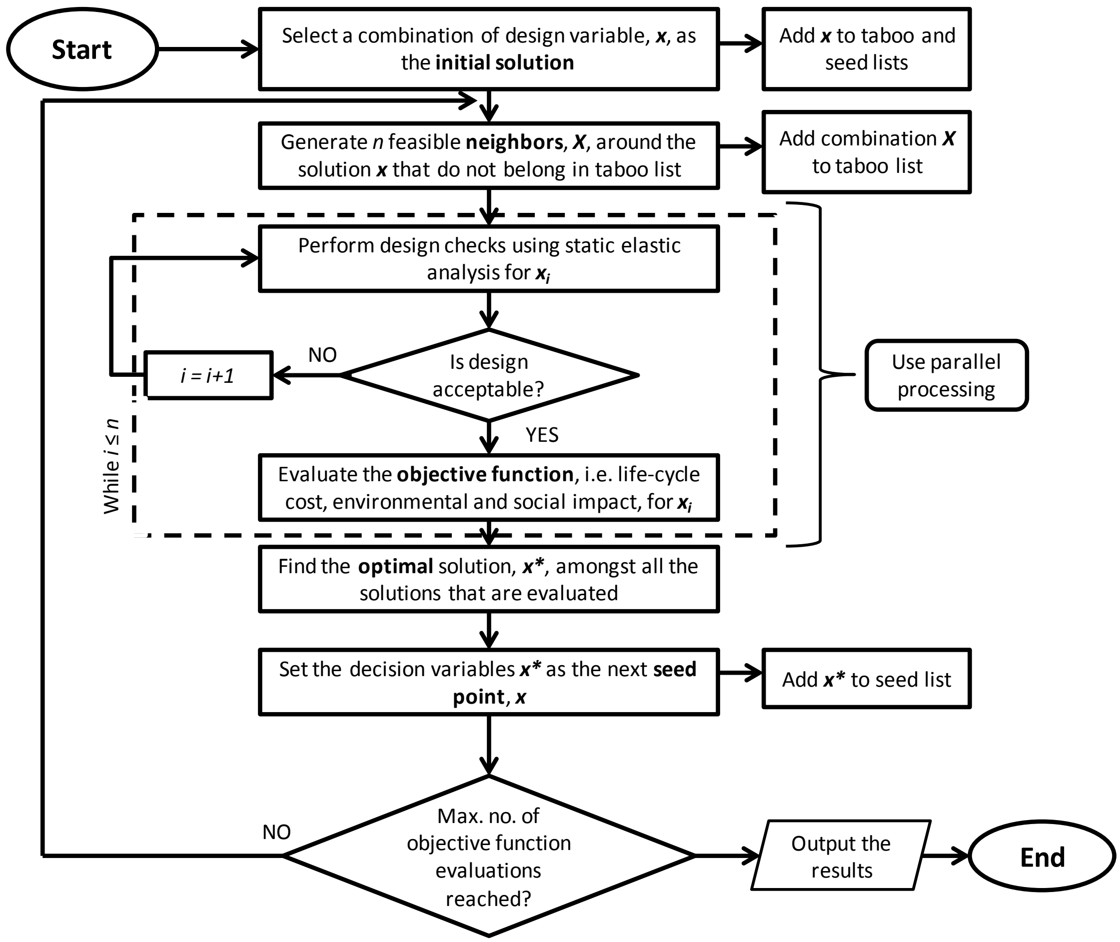

Figure 8: Flowchart of the structural optimization approach

To explore the search space, at each iteration TS selects a set of neighboring combinations of decision variables using some optimal solution as a seed point. Usually, a portion of the neighboring points is selected randomly to prevent the algorithm being trapped at a local minimum. TS algorithm uses a number of memory structures to keep track of the previous evaluation of objective functions and constraints. The most important memory structure is called the tabu list, which temporarily or permanently stores the combinations that are visited in the past. TS excludes these solutions from the set of neighboring points that are determined at each iteration. The existence of the tabu list is crucial for the optimization problem considered here because the evaluation of objective functions and/or constraints are computationally costly.

3.6 Conclusions

The comprehensive framework for seismic sustainability assessment was developed in this Chapter. All three sustainability components: economy, environment and society were addressed. LCA was used for quantifying sustainability in terms of various metrics. LCSPA is proposed as an integrator tool that relates seismic hazard and structural performance to give a probabilistic estimation of the amount of damage. LCCA and LCEIA are used to evaluate the environmental and economic impacts. Since no single social impact index is available, downtime is recommended as a measure of social impact. Finally a multi-level multi-objective optimization methodology is developed which uses the sustainability components as objective functions to be minimized.

4. A CASE STUDY ON SUSTAINABILITY ASSESSMENT OF RC BUILDING

In this section, the sustainability assessment framework developed in Chapter 3 is applied to a case study RC building. At first, a finite element model of the building frame is developed, and nonlinear structural analysis is performed using various ground motions representing different hazard levels. Sustainability components corresponding to the initial and end-of-life phases are derived from the design parameters of the building. PBEE has been adopted to obtain the service life losses in terms of repair cost, repair environmental impact, and downtime. Afterwards, a multi-level multi-objective optimization approach is applied to investigate the selection of design parameters while minimizing the total cost, the total environmental impact and the total time for construction and repair. Finally, results are compared between optimal and non-optimal designs with a view to demonstrating the contribution of service-life sustainability metrics on the total economic, environmental and social impact.

4.1 Seismic Hazard Analysis and Earthquake Ground Motions

This section describes the methodology for selection of ground motions to be used in nonlinear structural analysis for performing LCSPA of the case study RC frame. For this reason, a geographical location for the structure is selected. By accommodating the different site-specific factors such as nearby faults and their mechanisms, source to site distances, soil properties, and the magnitude and recurrence intervals of earthquakes, seismic hazard is developed. Three seismic hazard levels are defined with 75, 475 and

2475 YRP in terms of uniform hazard spectra (UHS). Seven compatible ground motions corresponding to each of these three hazard levels are selected for LCSPA.

4.1.1 Definition of Seismic Hazard

In the probabilistic seismic hazard analysis, a specific site is selected to have a realistic representation of the hazard. For this purpose, a site located in downtown San Francisco, California, with coordinates 37° 46' 29.67" N, 122° 25' 10.12" W, is selected. According to NEHRP (FEMA 2003) scale shown in Figure 9(a), the site class is categorized as class D with shear wave velocity ranging from 180 m/sec to 360 m/sec. Several major fault systems affect the seismicity in the San Francisco bay area including San Andreas, San Gregorio and Hayward faults, as shown in Figure 9(b).

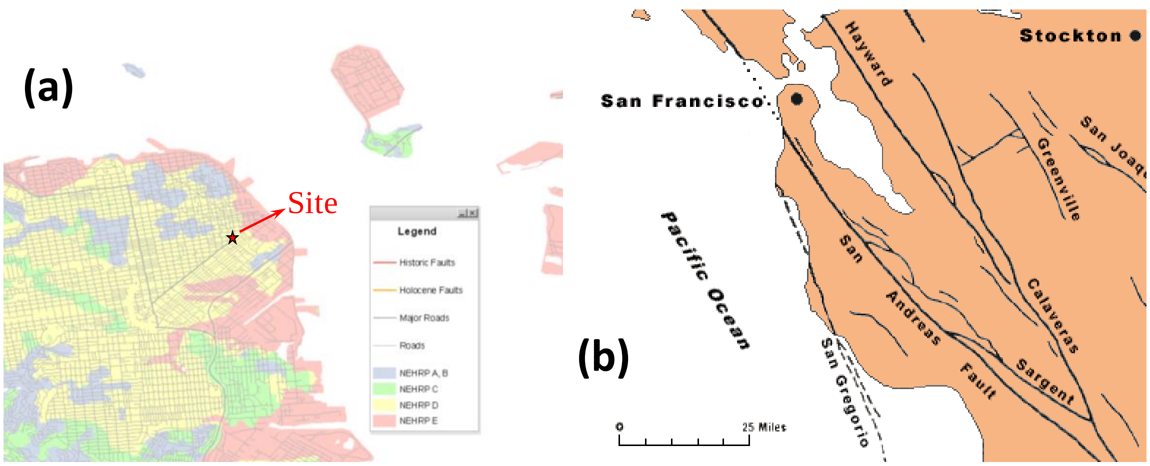

Figure 9: (a) Soil profile in site location (b) Major faults near the selected sites (USGS 2009; USGS 2009)

4.1.2 Selection and Spectrum Matching of the Earthquake Ground Motions

An online applet developed by (USGS 2009) is used to produce UHS corresponding to five different hazard levels. UHS are generated [Figure 10(a)] only at three previously mentioned return periods (i.e., 75, 475 and 2475 years) since they correspond to different structural performance and design limit states levels explained in Section 3.2.1. Site-specific hazard curves in terms of PGA as an intensity measure are written in the mathematical:

$$P(PGA) = c_1 . e^{c_2 . PGA} + c_3 . e^{c_4 . PGA}. \quad (7)$$

Here c_1 through c_4 are constants to be evaluated using curve-fitting techniques for the developed hazard curves developed. The fitted curves using Equation (7) are shown in Figure 10(b). The major benefit of representing hazard curve in terms of such a mathematical form is that POE of PGA can be analytically differentiated which simplifies the loss assessment step of PBEE, which will be discussed later on.

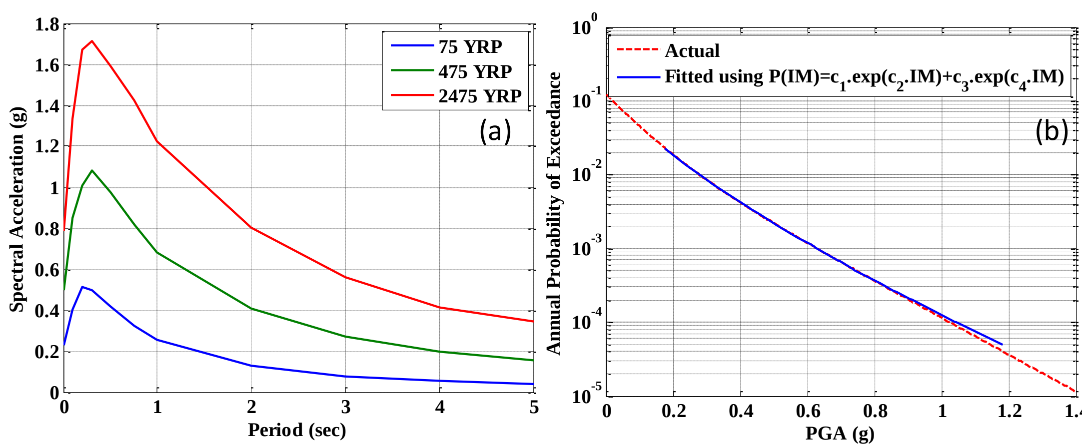

Figure 10: (a) Uniform Hazard Spectrum (UHS), (b) Hazard Curve

PBEE for the selected site is conducted for the above mentioned three hazard levels. FEMA 356 (FEMA 2000) recommends using five to seven ground motions for each hazard level for performing nonlinear analysis. Therefore, seven ground motions are selected from PEER Next Generation Attenuation (NGA) relationship database (PEER 2005). Spectrum matching is performed in order to make the ground motions compatible with the UHS using the modified version of the RSPMatch software (Abrahamson 1993) described in Hancock et al. (2006). Spectrum matching with UHS was only done for a period of 0-1 sec, since the fundamental period of structure is expected to be in this range irrespective of the structural design; while, the original spectra of the records are maintained for periods larger than 1 sec for excluding unrealistic low frequency oscillation. This approach of spectrum matching offers less record-to-record variability of structural response compared to acceleration scaling of records (Hancock et al. 2006; Al Atik and Abrahamson 2010). Detailed procedure of spectrum matching can be found in Gencturk (2011). The plot of response spectra of the selected ground motions before and after spectrum matching is provided in Figure 11(a), Figure 12(a) and Figure 13(a) for 75, 475 and 2475 YRP, respectively; while, the plot of acceleration time histories of sample original and spectrum compatible records are shown in Figure 11(b), Figure 12(b) and Figure 13(b), for the same return periods. The spectrum compatible time histories are used for nonlinear inelastic dynamic analysis.

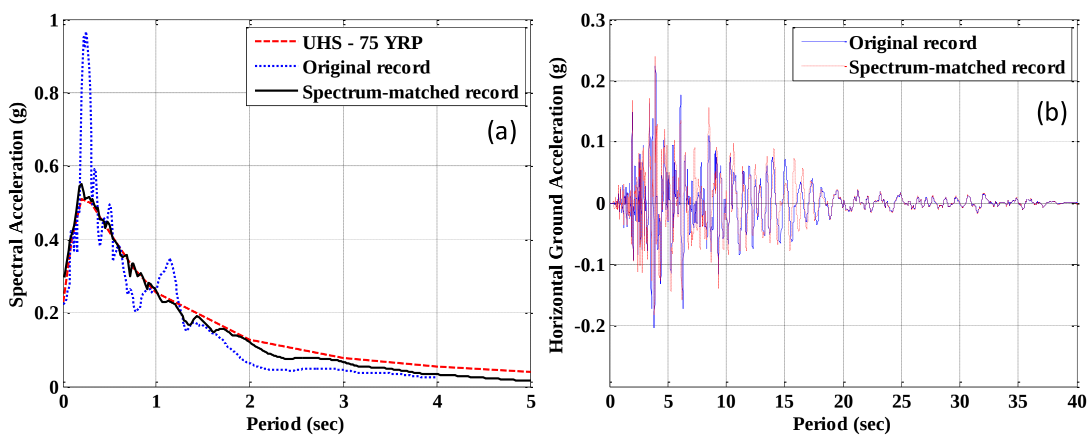

Figure 11: (a) Acceleration response spectrum of spectrum compatible ground motion, (b) Original and spectrum-matched ground motion – 75 YRP

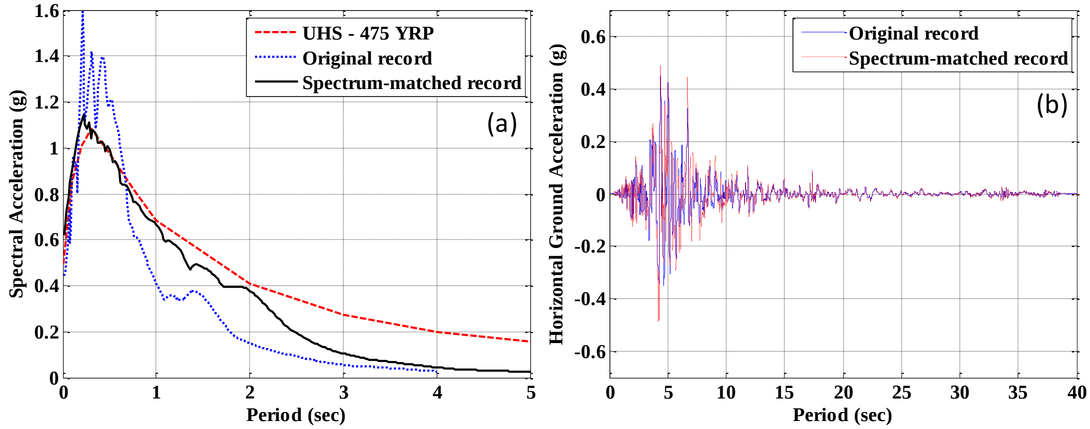

Figure 12: (a) Acceleration response spectrum of spectrum compatible ground motion, (b) Original and spectrum-matched ground motion – 475 YRP

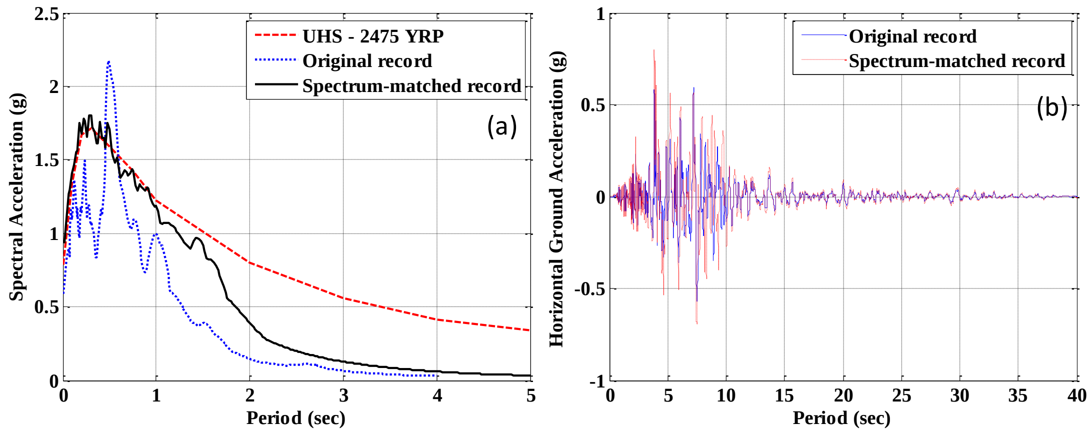

Figure 13: (a) Acceleration response spectrum of spectrum compatible ground motion, (b) Original and spectrum-matched ground motion – 2475 YRP

4.2. Modeling, Structural Analysis and Additional EDP Generation

4.2.1 Selection of Layouts and Initial Design of Structural Frame

The hypothetical four-story three-bay RC moment resisting frame shown in Figure 14 is chosen for performing LCA and design optimization. The space-frame is a part of three-dimensional RC building designed according to IBC-2003(ICC 2003), ASCE 7-02 (ASCE 2002), and ACI 318-02(ACI 2002). The design is adopted from Haselton and Diererlein (2007), and is briefly summarized here.

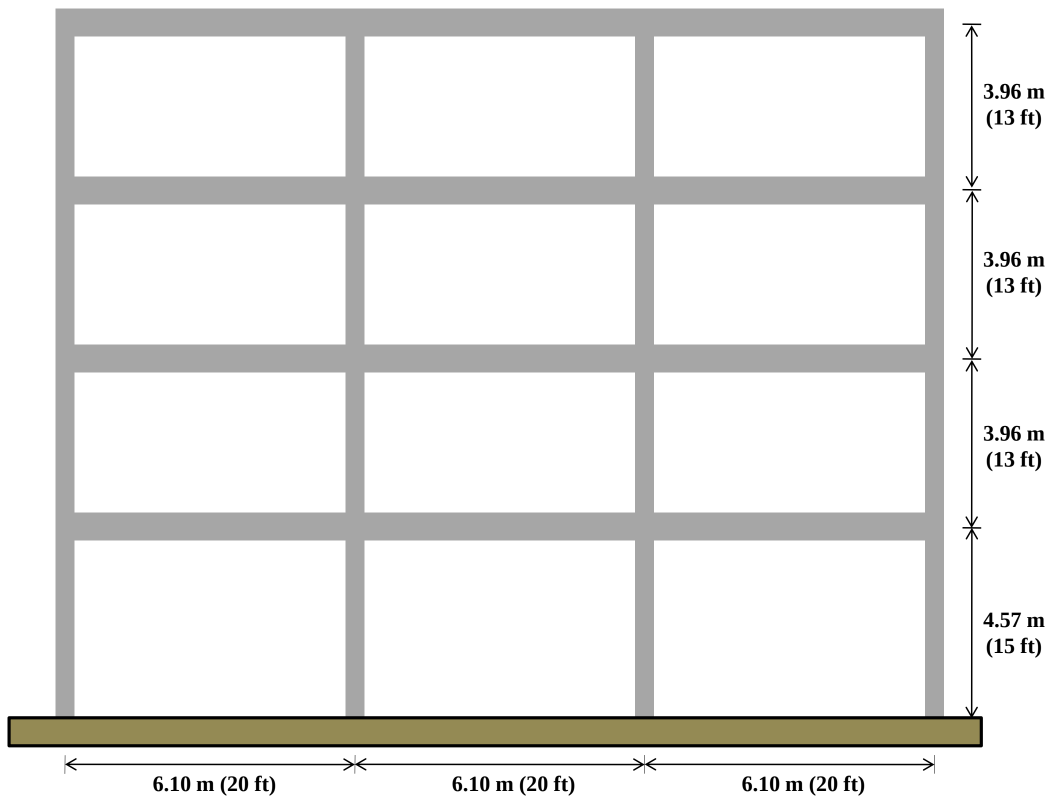

Figure 14: The considered structural frame for optimization

The material used for design and construction is reinforced concrete with an ultimate strength of 241 N/mm^2 (5 ksi) and rebars with yield strength of 414 N/mm^2 (60 ksi). In the original design, initial column sizes are 558.8 mm x 558.8 mm (22 in x 22 in) with total longitudinal reinforcement ratios varying from 1.13% to 1.63% depending on the story level and column type (exterior or interior). Typically, reinforcement ratios are higher in the interior columns. All beams are assigned a cross sectional dimension of 558.8 mm x 609.6 mm (22 in x 24 in). Reinforcement ratios at tension and compression sides of beams are reduced gradually with increasing floor level from 0.83% and 0.43%

(first floor level) to 0.45% and 0.32% (roof level), respectively. Beam stirrups and column stirrups are spaced at 127 mm (5 in) with total reinforcement ratios of 0.33% and 0.7%, respectively. The design inter-story drifts at each story ranged between 0.6% and 1.2%.

The column and beam sizes were selected as 635 mm x 635 mm (25 in x 25 in) and 381 mm x 635 mm (15 in x 25 in), respectively in the finite element model. The reinforcement ratio of all columns were chosen as 0.01, while that of the bottom two stories and top two stories were assigned to be 0.015 and 0.01, respectively. The stirrups were designed according to the code requirements. The alteration of these design parameters was done so that they are kept in line with the decision variable chosen for structural optimization as discussed in Section 4.5. In structural optimization, design variables (usually section sizes and reinforcement ratios) are selected from a set of discrete values. After finding the objective functions for a set of design variables, the optimization algorithm assigns the next set of design variables from the neighboring points. This requires that, the initial design comprises design variables from the solution space.

4.2.2 Linear Elastic Analysis for Design Check and Capacity Assessment

The gravitational loads for the considered space (interior) frame include a floor dead (including self-weight) and live loads of 8379 N/m^2 (175 psf) and 2394 N/m^2 (50 psf), respectively. The equivalent lateral load method, which is one of the recommended procedures of ASCE 7-10 (2010), is used for defining the earthquake loads in elastic analysis. Lateral loads are computed according to ASCE 7-10 section 12.8.1. The response modification factor (R), overstrength factor (Ω_0), and deflection amplification factor (C_d) corresponding to a special moment resisting frame, i.e., 8, 3 and 5.5, respectively, are used. Effective seismic weight of the frame is taken as the full dead load plus 25% of the live load. Based on the seismic design coefficients and the seismic weight, design seismic base shear is found to be 360.3 kN (81 kips). The total base shear is then distributed at each floor level by assuming an inverted triangular (code suggested) distribution.

At first, the code compliance of the initial design is checked with ACI 318-08 (2008) and IBC 2009 (ICC 2009) using linear static analysis. All the load combinations (including the seismic effects) stipulated in these regularity documents are taken into account. Additionally, P-Delta effects are accounted for in the analysis and design checks. The structural capacity is not measured based on a specific response quantity but it is checked based on serviceability and strength criteria. The modified initial design was found to comply with all the provisions of the design codes.

Structural capacity is then evaluated using static pushover analysis. An inverted triangular distribution is applied over the height of the building. The roof displacement is gradually increased and the base shear corresponding to the displacement is calculated. Structural capacity or collapse state is defined according to the ultimate deformation given by Park (1988) as the post-peak displacement or drift when the load carrying capacity (base shear) has experienced a small reduction of 10-15%. The pushover curve is plotted in Figure 15, where the ultimate structural capacity is found as 7.62% based on 10% post peak reduction.

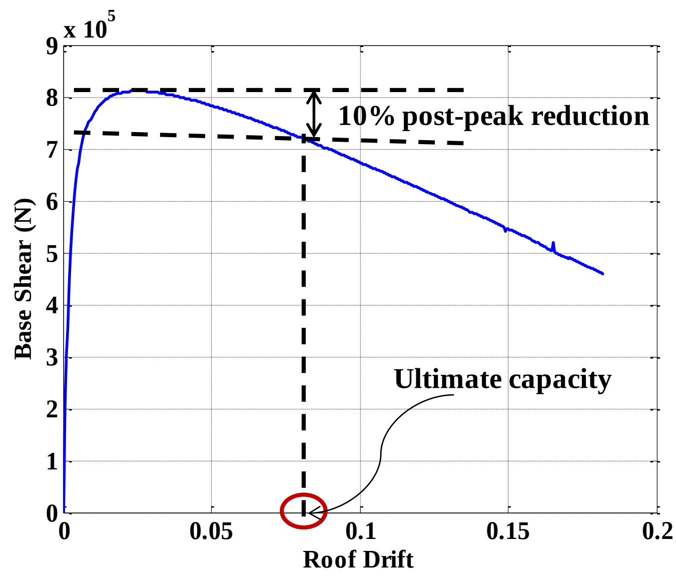

Figure 15: Pushover curve for capacity assessment

4.2.3 Nonlinear Inelastic Analysis for Response Evaluation

Earthquake demand is evaluated through a nonlinear inelastic dynamic time history analysis using the fiber-based finite element analysis program ZEUS NL (Elnashai et al. 2010). The structural frames are modeled using displacement-based beam-column elements with cubic shape functions (Izzuddin and Elnashai 1993).

Concrete (Martínez-Rueda and Elnashai 1997) and reinforcing steel (Ramberg and Osgood 1943) are modeled using the existing models in ZEUS NL materials library. Geometric nonlinearity is taken into account in the dynamic analysis. A response history analysis is performed under each of the spectrum compatible ground motions developed in hazard analysis step. Since the building is regular and symmetric, only two-dimensional model of the frame is developed for structural assessment.

As described earlier in Section 3.2.2, the earthquake demand is measured in terms of several EDPs such as maximum compressive and tensile strains of unconfined concrete (ε_{uc}), confined concrete (ε_{cc}) and reinforcing steel (ε_s) and maximum interstrory drift. While the strains are captured at all elements, interstory drift is selected as an additional EDP for columns because of being closely related to the development of P-Delta instability (a system level indicator), and to the amount of local deformation imposed on the vertical elements and beam column connections (component level indicators). Without using any performance group (e.g., all the columns of the same story), as opposed to other studies, the selected EDPs are captured at each element for each ground motion. The selection of EDPs was such that they could effectively capture different criteria that define the damage states as described in Section 3.2.3. A sample realization EDP matrix for a beam element is shown in Table 9 for a hazard level of 50% POE in 50 years hazard level (75 YRP).

Table 9: Sample EDP Matrix

EDP	GM1	GM2	GM3	GM4	GM5	GM6
$(\varepsilon_{uc})_t$	0.002795	0.003755	0.002035	0.003853	0.002325	0.002608
$(\varepsilon_{uc})_c$	0.000648	0.000724	0.000533	0.000685	0.000796	0.000638
$(\varepsilon_{cc})_c$	0.000359	0.000355	0.000305	0.000424	0.000603	0.000373
ε_s	0.002647	0.003563	0.001923	0.003656	0.002272	0.002467

4.2.4 Additional EDP Generation

Adequately large numbers of realization of EDPs are essential not only to obtain meaningful statistical data but also to properly account for uncertainty and its propagation from one step to another. This is hampered due to unavailability of representative ground motions and high computational demands of a dynamic analysis. In order to overcome this obstacle, Yang et al. (2009) proposed a methodology for generating additional correlated EDP vectors. Their procedure is based on the assumption that the EDP realizations of a given element exhibit jointly lognormal distribution. Based on this formulation, 500 EDP vectors are generated for each element under each hazard level. A sample of additionally generated EDPs from the original values is provided in Table 10. The original and the additionally generated EDPs, for one of the beams of corresponding to 75 YRP, are compared in terms of mean and median values in Table 11. The values show good correlation which proves that additionally generated EDPs possess the same distribution as the original EDPs, and can be used for the damage assessment as described below.

Table 10: Additionally Generated EDPs

EDP	1	2	3	4	5	6	-	500
$(\varepsilon_{uc})_t$	0.00323	0.00306	0.00706	0.00339	0.00273	0.00335	-	0.002407
$(\varepsilon_{uc})_c$	0.00086	0.00058	0.00115	0.00069	0.00079	0.00060	-	0.000616
$(\varepsilon_{cc})_c$	0.00047	0.00029	0.00063	0.00041	0.00062	0.00032	-	0.000375
ε_s	0.00313	0.00286	0.00690	0.00322	0.00266	0.00315	-	0.002276

Table 11: Comparison between Original and Additionally Generated EDPs

EDP	Mean		Median	
	Original	Additional	Original	Additional
$(\varepsilon_{uc})_t$	0.002919	0.002919	0.002701	0.002791
$(\varepsilon_{uc})_c$	0.000669	0.000669	0.000666	0.000663
$(\varepsilon_{cc})_c$	0.000404	0.000404	0.000366	0.000394
ε_s	0.002777	0.002777	0.002557	0.002661

4.3 Damage Assessment

In this step, EDP values are translated into DSs. Only structural damage is considered since the performance of nonstructural components is outside the scope of this study. Different threshold values of selected EDPs are used to define damage states as mentioned in Section 3.2.3. In order to determine DSs for a given element, fragility curves are developed based on the threshold values provided in Table 3. A lognormal distribution is assumed and dispersion (logarithmic standard deviation) values of 0.3 and 0.4 are used for concrete and rebar related EDPs, respectively. This assumption was made based on the fact that concrete inherently exhibit more variability compared to steel due to material microstructure and fabrication conditions. Since several EDPs are used to define DSs, more than one fragility curve is obtained as shown in Figure 16.

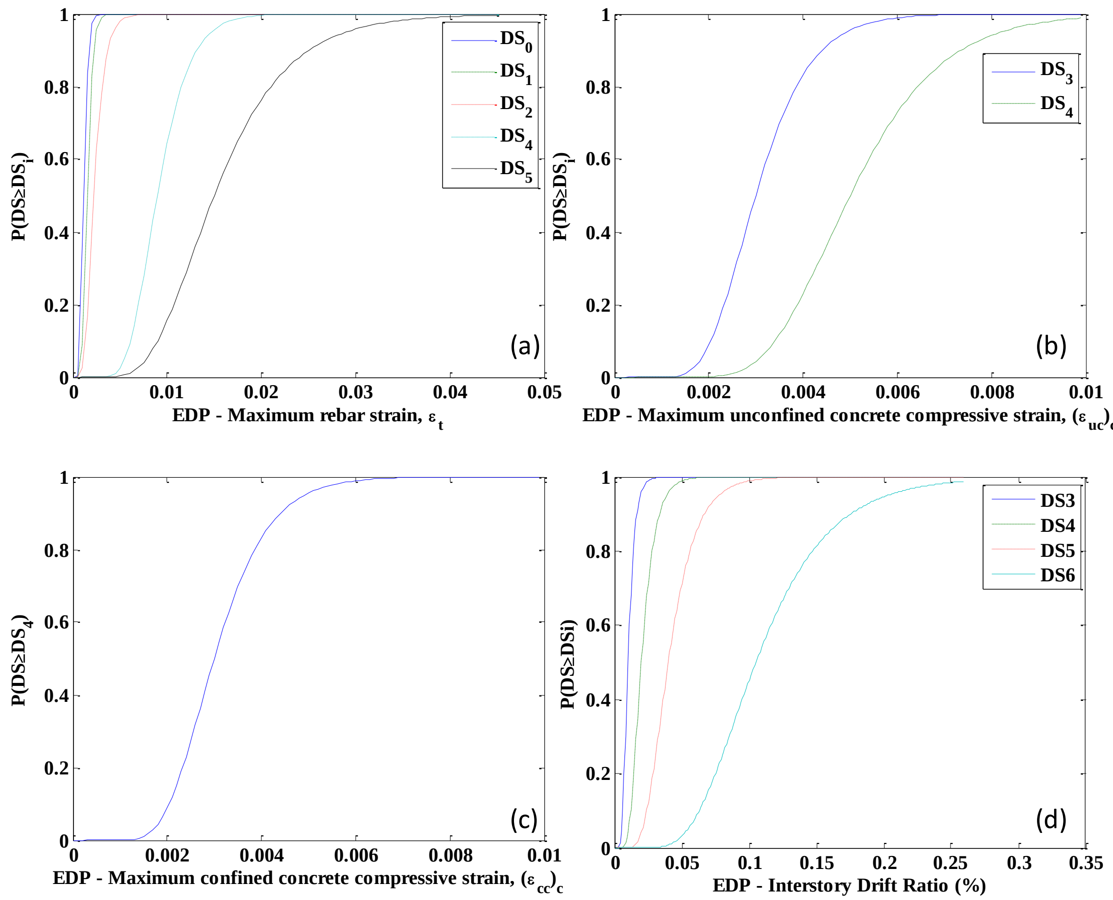

Figure 16: Fragility curves in terms of (a) rebar strain, (b) unconfined concrete compressive strain, (c) confined concrete compressive strain, and (d) interstory drift ratio

The probability for an element to be at a certain damage state is calculated from the fragility function:

$$F_{ds}(edp) = P(DS \geq ds|EDP = edp), \tag{8a}$$

$$F_{ds}(edp) = \Phi(\frac{ln(edp/x_m)}{\beta}), \tag{8b}$$

where Φ is the standard normal cumulative distribution function, x_m denotes median values of EDPs and β is the logarithmic standard deviation. If U is a number between 0 and 1, produced by a uniform number generator, then the element 'j' will be in the 'i'th DS if P is greater than U. Using this approach, every additionally generated EDPs is assigned a DS.

4.4. Loss Estimation

The main reason for performing LCSPA through PBEE is to calculate losses in terms of the basic sustainability components: cost, environmental and social impact. For this reason, the DVs are selected such that monetary, environmental, and temporal losses, occurred due to restoration of the strength and stiffness of structural components, are properly accounted for. Hence DVs are presented in terms of the cost, environmental, and social impact due to service life phase. While cost and environmental impact are expressed as dollar and EPS, the repair time is selected as an indicator of social impact.

Damage assessment provides useful results in terms of probabilities that an element will be in a certain damage state for a given hazard level. DSs are translated into DVs using the methodology described in Chapter 3. The repair cost, repair environmental impact, and repair time values corresponding to different hazard levels fit well to lognormal distributions. The POE of a certain value of DV is obtained from the cumulative distribution function (CDF) of the realization of the DV. The mean annual rate of exceeding a given value of DV is obtained by integrating the complementary

CDFs. POE and mean annual rate of exceedence of repair cost, repair environmental impact and repair time is plotted in Figure 17 through 19. The area under the loss curves represents the total annual losses. For the building service life of 50 years, the total life-time seismic repair cost, environmental impact and time are found to be \$ 67,089, 4,976,347 EPS, and 339 Man-hours, respectively.

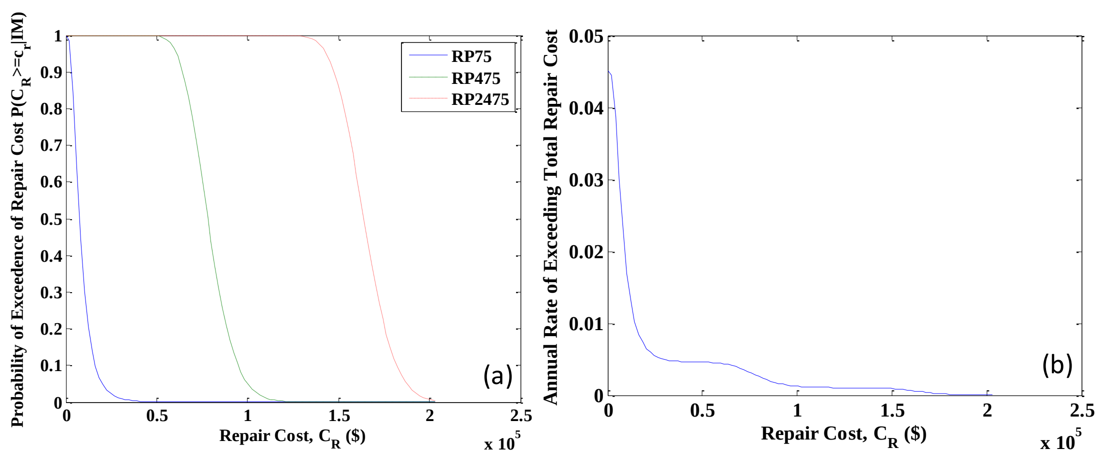

Figure 17: (a) POE of repair cost, (b) mean annual rate of exceedance of repair cost

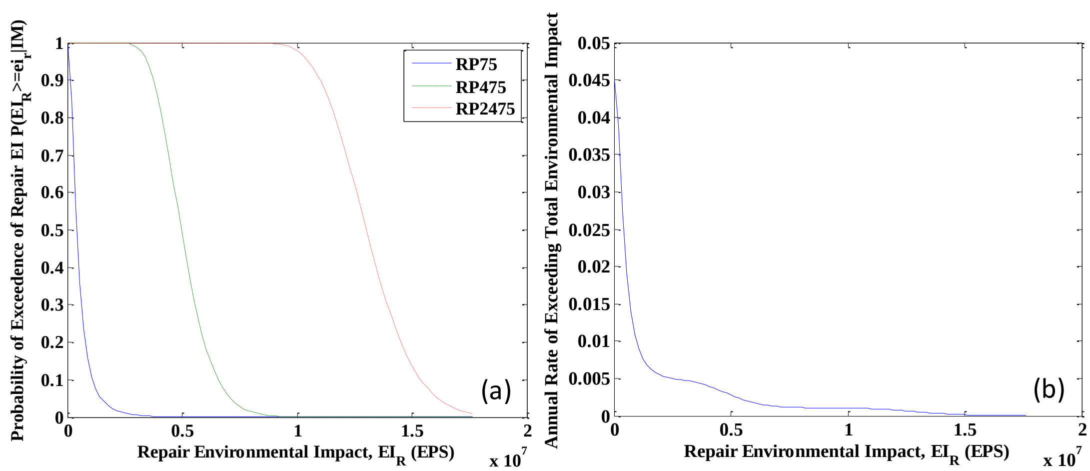

Figure 18: (a) POE of repair environmental impact, (b) mean annual rate of exceedance of repair environmental impact

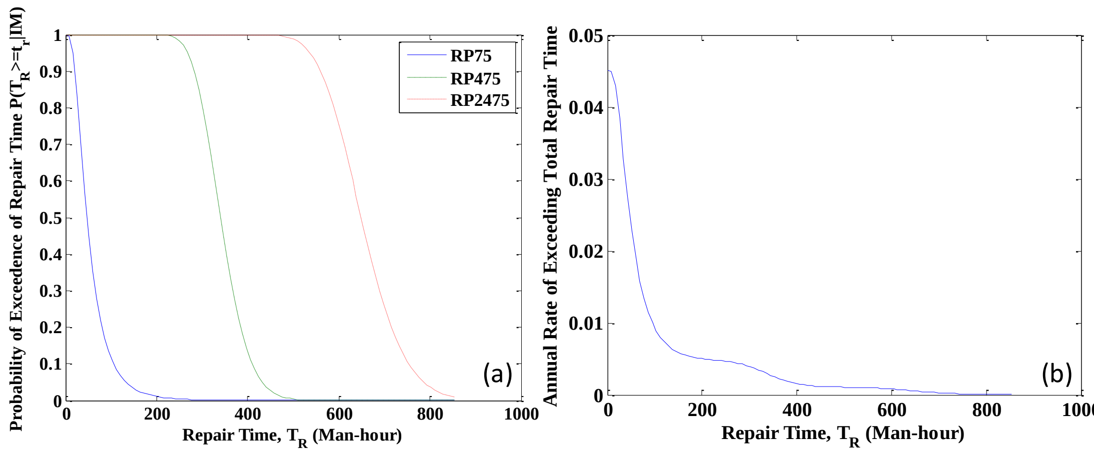

Figure 19: (a) POE of repair time, (b) mean annual rate of exceedance of repair time

4.5. Structural Optimization

The three major features of any optimization problem are decision variables, objective functions, and constraints. For the case study RC frame, the decision variables are chosen as the section sizes and reinforcement ratios of the structural members subjected to the constraints given in seismic design documents (ACI 2008). The total cost, total environmental impact and total time for construction and repair are selected as objective functions to be minimized.

In order to keep the search space within the allowable limits, the number of decision variables is reduced with some realistic assumptions. While most of these assumptions comply with the seismic resistant construction practices for RC structures, some are used to simplify the optimization method. The assumptions are:

i. The topology of the frame is kept the same. Only the section dimensions and reinforcement ratios are considered as design variables.

ii. All columns of the frame are taken as square sections.

iii. Reinforcement ratios are constant throughout a beam or a column. Designing with reduced reinforcement area at the low-moment region is not employed.

iv. Beam width in all floors is taken as a constant value of 381 mm, thus is excluded from the design variables.

v. Beam depth and reinforcement ratios are changed every two floors.

vi. Shear reinforcement is determined according to design code based on elastic analysis, and not considered as a design variable.

vii. All beams and columns have a predetermined reinforcement configuration shown in Figure 20 The area of each rebar, calculated from the reinforcement ratio and section size, may not correspond to a commercially available rebar size.

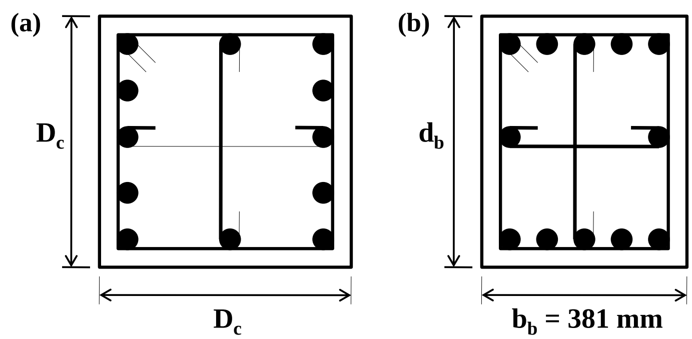

Figure 20: Typical cross sections of (a) columns and (b) beams.

Based on the assumptions above the number of design variables is reduced to seven, which are column width, reinforcement ratios of exterior and interior columns, depth and reinforcement ratio of first two story beams, and depth and reinforcement ratio of top two story beams. Discrete, practical values are used for each of these design variables as provided in Table 12. The bounds of these values comply with ACI 318-08

(2008) and ACI 318-11 (2011). All possible combinations of these design variables generate 279,936 cases, which set up the search space for this optimization problem.

Table 12: Design Variables and Ranges for the Considered Structural Frames

Design Variables	**Values**
Width of columns (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio of external columns	0.01, 0.02, 0.03, 0.04, 0.05, 0.06
Reinforcement ratio of internal columns	0.01, 0.02, 0.03, 0.04, 0.05, 0.06
Depth of first two story beams (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio (first two story beams)	0.005, 0.01, 0.015, 0.0175, 0.02, 0.025
Depth of top two story beams (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio (top two story beams)	0.005, 0.01, 0.015, 0.0175, 0.02, 0.025

The objectives of the optimization problems are highly nonlinear because of using inelastic dynamic analysis for response and demand evaluation, and the derivatives of these objectives with respect to selected design variables are discontinuous. In addition, the decision variables take discrete values. These reasons restrict the use of gradient-based optimization algorithms. Hence, TS algorithm as discussed in Section 3.5.3 is used for optimization.

4.6 Life-Cycle Optimization Results and Comparisons

The results from elastic and inelastic analysis are shown in the solution space in Figure 21. Each of the points in the solution space represents a combination of design variables that has satisfied the design checks according to ACI 318-08 (2008). For multi-objective optimization, usually no single optimal solution is obtained. Instead, a set of equivalent optimal solution forming a Pareto front is obtained, from which the decision makers can choose based on the priorities of the structure. In order to get a Pareto set of optimal solution, the objectives should be competing with each other. For example, minimizing initial cost and maximum interstory drift (which are two competing objectives) provides a Pareto set of solutions (Gencturk and Hossain 2013). On the contrary, directly related objectives, such as those in the current study, do not produce a set of optimal solution as it is seen in Figure 21.

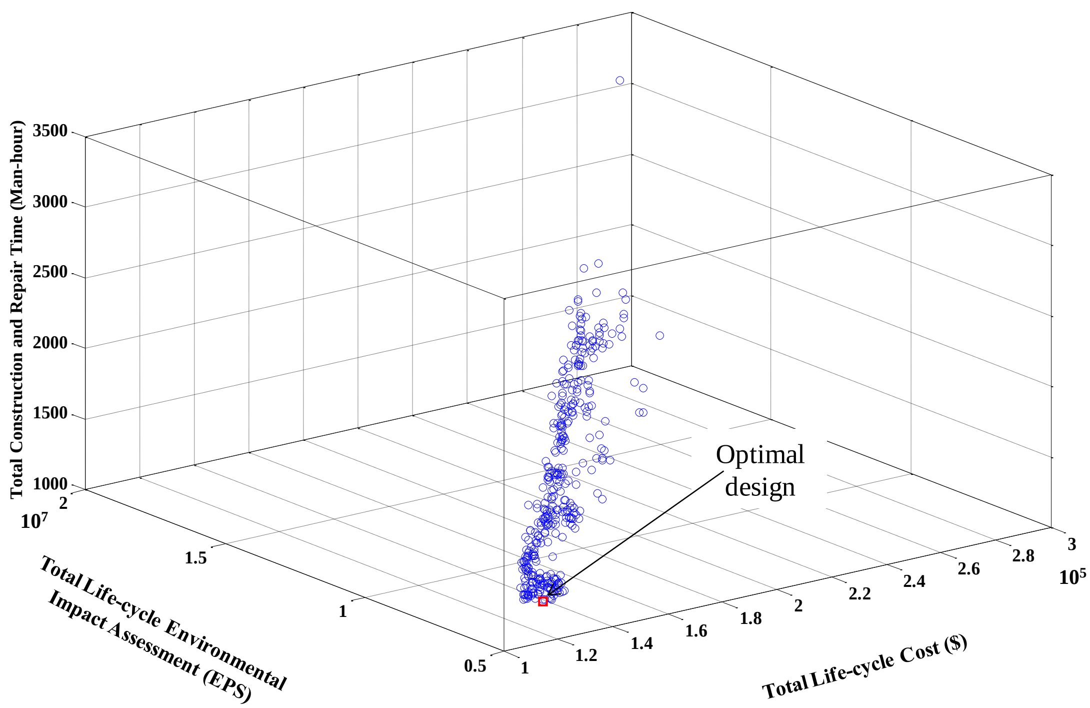

Figure 21: Results of multi-objective optimization in the solution space

It is seen that there exists significant variation between the three sustainability components for various code-conforming designs. This proves that selection of design variables is very important in order to achieve the most sustainable solution. Another important finding is that there exist a number of cases where the total cost, environmental and social impacts are substantially high. This can be attributed to significant damage, element collapse, or global collapse of the building under frequent or design earthquakes. The objectives are plotted in pairs in Figure 22 for better visualization.

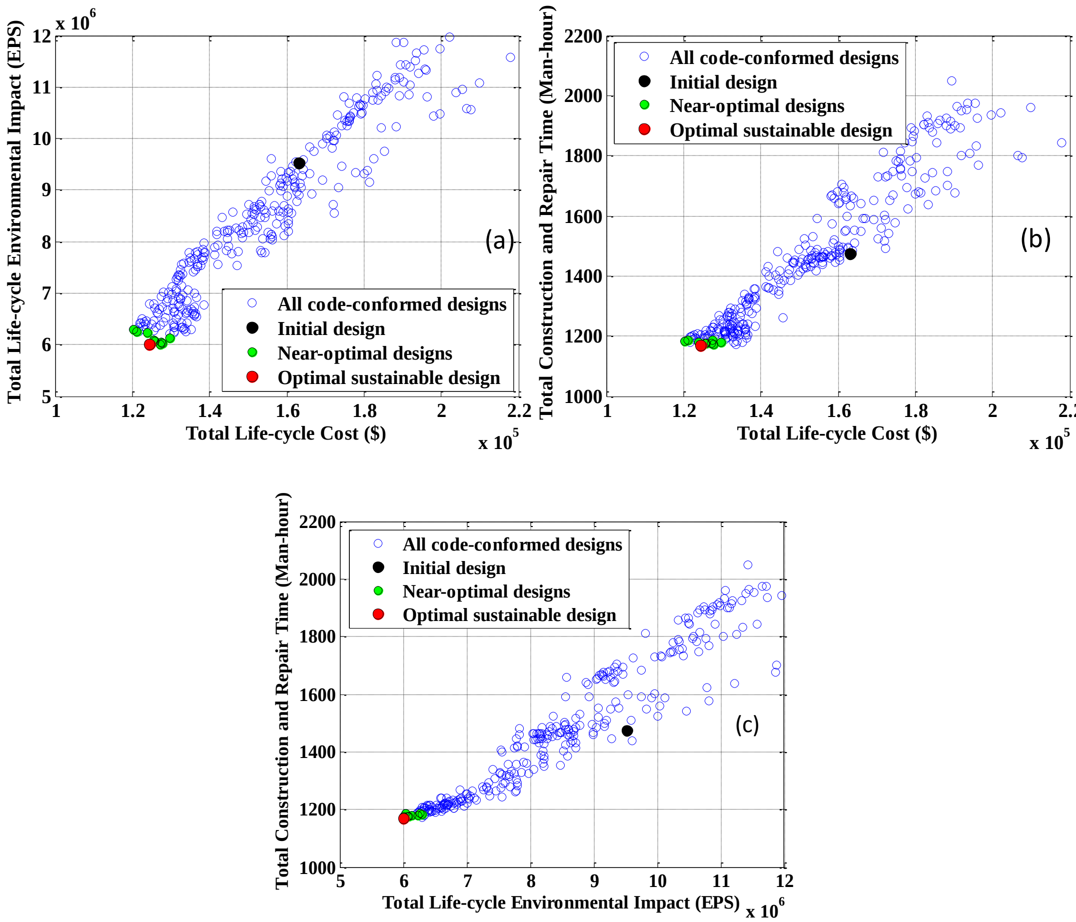

Figure 22: Results of multi-objective optimization in the solution space (a) cost vs. environmental impact, (b) cost vs. total time, (c) environmental impact vs. total time

Among all the alternative designs, eight designs close to the optimal sustainable design are selected in order to assess the contribution of each design variable to the total cost, environmental impact and time for construction and repair activities. The selected solutions, along with the initial and optimal design, are identified in Figure 22 and corresponding decision variables are shown in Table 13 along with the total impact values.

Table 13: Comparisons of Alternative Designs

Design Type	Design Variables							Total Cost ($)	Total Environmental Impact (EPS)	Total Construction and Repair Time (Man-hour)
	Depth and width of columns (mm)	Reinforcement ratio of external columns	Reinforcement ratio of internal columns	Depth of first two story beams (mm)	Reinforcement ratio of first two story beams	Depth of top two story beams (mm)	Reinforcement ratio of top two story beams			
Initial	635	0.01	0.02	635	0.015	635	0.01	163,053	9,524,913	1,473
Optimal	508	0.03	0.03	381	0.01	508	0.005	124,368	6,006,574	1,169
Near-optimal	508	0.03	0.04	381	0.01	508	0.005	125,718	6,067,502	1,176
	508	0.03	0.03	381	0.015	508	0.005	127,304	6,009,336	1,177
	508	0.03	0.03	381	0.01	508	0.01	125,507	6,092,402	1,176
	508	0.02	0.03	381	0.0175	508	0.005	127,844	6,015,909	1,172
	508	0.02	0.03	381	0.015	508	0.005	127,456	6,047,809	1,185
	508	0.02	0.03	508	0.02	508	0.01	123,887	6,239,815	1,179
	508	0.03	0.03	635	0.015	508	0.02	120,308	6,291,800	1,182
	508	0.03	0.03	635	0.015	508	0.0175	121,225	6,260,758	1,187
	508	0.03	0.03	381	0.0175	508	0.005	129,634	6,138,418	1,178

It can be seen that a number of design combinations provide comparable results to the optimal solution. For all these design combinations which are approaching the optimal solution, the dimensions of all members except for the depth of bottom two stories were identical. However, the reinforcement ratios of these elements varied. This observation can be explained as follows. Reduced values of the sustainability metrics are obtained from initial (production and construction) life-cycle phases because of using a less amount of steel. On the other hand, reinforcement ratios of beams and columns are important design parameters that regulate the structural performance under seismic loads. Lower reinforcement ratio might have resulted in more damage, which increased the life-time impacts.

A breakdown of cost, environmental impact and time, shown in Figure 23, gives good idea about the contribution of different life-cycle phases to the total cost, environmental impact and time. While the material production cost for initial and optimal designs were close, the construction and repair cost was significantly high. At the end, the total life-cycle cost of optimal design was almost three-fourth of the initial design. Note that, the repair cost of the optimal design was almost as much as the initial material production and construction cost. In spite of the high repair cost, it was still found to be the most cost effective. This proves that although conservative design may reduce the damage experienced by the structure throughout its service life, it does not necessarily ensure the optimal cost-effective solution.

A similar conclusion as for the cost analysis can be drawn for environmental impact as well. The only difference in case of environmental impact is that construction phase has relatively smaller contribution compared to material production and repair phase. The high repair environmental impact indicates that the structures experienced significant amount of damage and possibly element collapse during its service life, and environmental impact was mostly associated with the production of materials used for repair. Repair time is found to be relatively smaller than the total construction time for both initial and optimal design. The time corresponding to end-of-life phase is relatively high, which is because instead of performing complete demolition, the elements are broken down to small pieces and carried to sorting plants. This step is necessary for separating recyclable materials from the rest of the demolished materials.

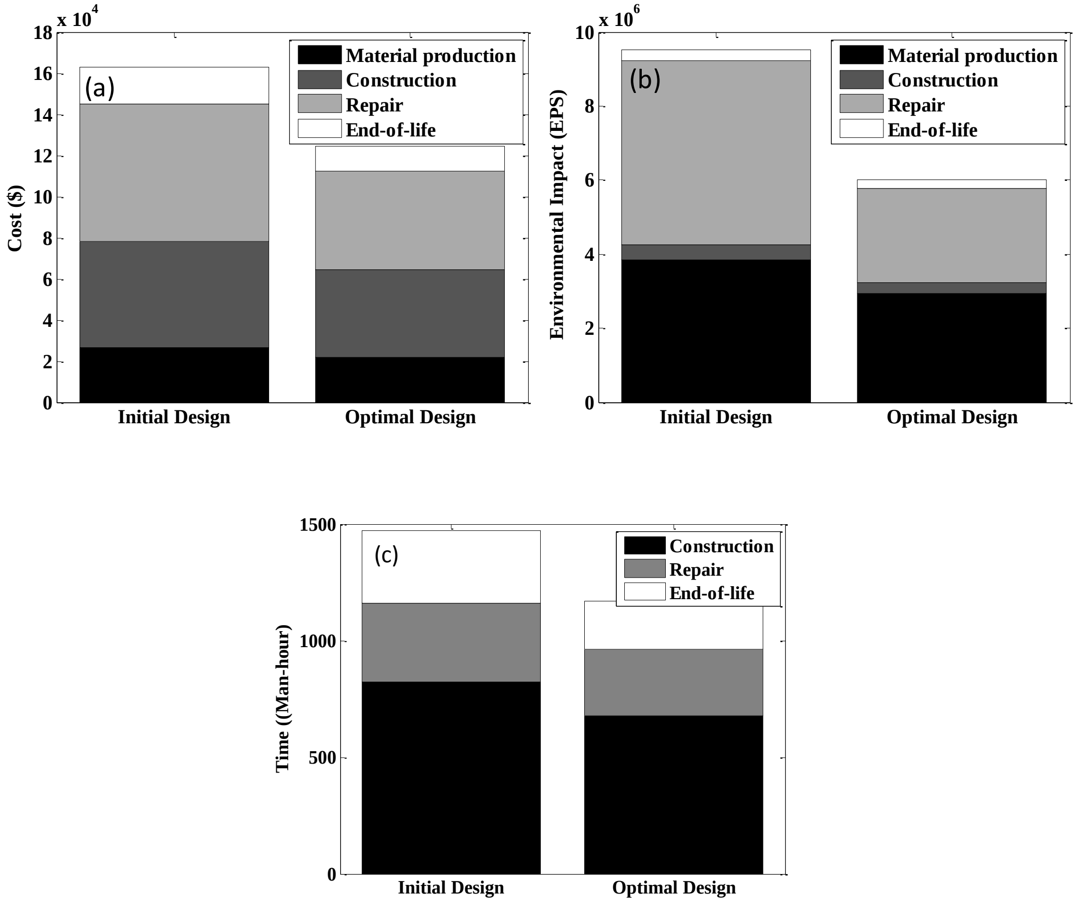

Figure 23: Comparison of cost, environmental impact and time the two designs

The initial cost can be further broken down into different processes associated with material production and construction. The breakdown of initial cost is shown in Figure 24. Both designs show considerably high values for production and construction of formworks. This may be due to high labor cost coupled with the assumption that forms are not reused during construction.

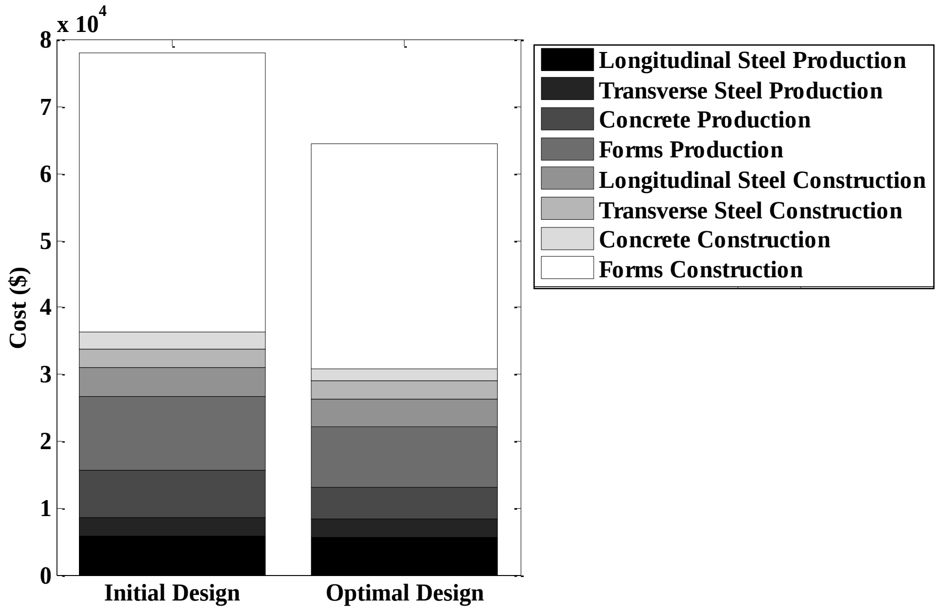

Figure 24: Breakdown of initial costs for both designs

4.7 Conclusions

This chapter applied the sustainability framework developed in Chapter 3 to a case study RC building frame. Hazard analysis is performed by matching available ground motion records with the location of the building. Nonlinear structural analysis is performed using finite element methods. The fragility curves are calculated and used to evaluate the damage states of the building. Then, the seismic loss is estimated in terms of cost, environmental impact and repair time corresponding to service life-cycle phase. By incorporating the initial and end-of-life phases, total values of sustainability components are evaluated. Structural optimization is performed with an aim to simultaneously minimize these components. A comparison between all code-conforming solutions shows that greater sustainability can be achieved through the incorporation of present framework.

5. CONCLUSIONS

Sustainability in the construction sector has received immense importance over the years, where sustainable practices mostly included initiatives towards reducing harmful environmental emissions without much attention towards economic and social contexts. However, recent devastating earthquakes demonstrated that unsatisfactory structural performance can greatly impede sustainable development. Therefore, a framework has been presented in this study for comprehensive sustainability assessment of RC structures subjected to natural hazards. Special attention has been given to the repair of structural damage due to future earthquake events in addition to material production, construction, and demolition phases of the structure. Inventories of cost, environmental emissions, and time (social indicator) have been developed for RC buildings, and damage assessment has been performed using performance-based methodology accounting for uncertainties at each step of structural performance assessment. Environmental emissions have been further converted to environmental impacts by normalizing and weighting of environmental input-output flows, whereas total cost and total time for construction and repair have been taken as economic and social indicators, respectively. Structural optimization has also been included within the sustainability assessment framework for optimizing the design in terms of quantifiable sustainability components. The main findings are summarized in the following section along with the limitations of this research, and recommendations for future research are provided.

5.1 Findings

The developed sustainability assessment framework has been applied on a case study RC structural frame, and the sustainability components: total (life-time) cost, environmental impact, and time have been calculated using the proposed framework. Additionally, the sustainability components have been used as performance objectives in a multi-level structural optimization methodology. This approach has enabled achieving optimality in the sustainable seismic design. Different non-optimal variations of the case study structure have been obtained from structural optimization, while a unique optimal sustainable solution has been achieved with minimum total cost, environmental impact, and time. Comparative analysis between the optimal and several non-optimal solutions have demonstrated that the cost, environmental and social impacts in the use/repair phase of the structure can be as high as 40, 50, and 25% of the total life-time impacts, respectively, which has further reinforced the necessity of incorporating structural damage and loss assessment in sustainability quantification procedure for seismic regions.

Most non-optimal solutions have shown lower contributions of impacts due to repair phases compared to the optimal sustainable solution, which indicate that employing high initial impact designs can sometimes minimize future structural damage and repair efforts. However, it does not necessarily ensure reduced life-time impacts. This also justifies the approach of incorporating sustainability metrics as performance objectives into seismic resistant designs, rather than using strength and serviceability based objectives. Comparative assessment of alternative designs can assist in decision making process through using the proposed framework.

5.2 Limitations of the Current Research

The main reason for selecting a RC frame building in this study was its simplicity and the availability of inventory data in terms of cost, environmental and social impacts for RC structures. Due to this reason, certain features could not be incorporated in the case study. However, it is possible to address these issues by adopting suitable modeling techniques. Some of the shortcomings of the case studies are presented here.

- Today, structures located in seismic regions are installed with energy dissipation devices such as shear walls or bracing systems, and bare frames are rarely used to withstand seismic forces. Although advanced damage resistant features were not included in the case study building, the sustainability framework can be properly tuned to apply it for modern seismic-resistant structures by developing appropriate inventories of sustainability components.
- During the damage assessment of RC building, it was assumed that the elements possess adequate ductility; hence the damage experienced by the elements is only due to flexure. This assumption was made because of using simplified finite element models that is unable to capture complex deformations such as shear and torsion. However, shear and torsional deformation can be introduced in LCSPA function by incorporating detailed finite element method for structural modeling and response analysis.

5.3 Recommendations of Future Research

It is recommended that future research regarding the subject addresses the following issues:

- Development of a more realistic damage state quantification technique, which would provide better assessment of the local damage patterns. The condition of having multiple damage states of the same element at different locations can also be introduced since different locations of a large element may undergo different levels of engineering demands.
- Development of reliable damage cost and environmental input-output models that include costs associated with damage to nonstructural components, loss of contents, relocation.
- Consideration of uncertainty in cost, environmental and time models, corresponding to a given damage state.
- Development of complete social impact assessment tool that takes into account all socially sensitive parameters, including more rigorous assessment of death tolls and injuries. A social impact index can be developed by using appropriate weighting value on the individual social impact metrics.
- Development of methods for identifying design variables for optimization that are practical for engineers and that govern the seismic response. This is necessary to minimize the computational cost since the nonlinear analysis procedure is computationally very expensive. Inclusion of reinforcement topology as a design variable can be considered for better representation of actual design situation.

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APPENDIX

Table A1: Life-Cycle Impact Categories, Adopted from B-PATH (Stadel et al. 2012)

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
Air Emissions													
1,3 Butadiene		5.30 E-08	3.80 E-02				3.32 E-01	1.19 E+01	1.21 E+00	4.33 E+01			3.23 E+00
2,4-Dinitrotoluene							3.45 E+00	5.09 E-02	1.58 E+02	2.33 E+00			
2-Chloroacetophenone													
5-methyl Chrysene													
Acenaphthene									1.80 E-01	6.54 E+00			
Acenaphthylene													
Acetaldehyde							3.53 E-03	6.33 E-03	3.91 E+00	1.08 E+01			1.79 E+00
Acetophenone									7.55 E+00	1.85 E+00			
Acid (organic)													
Acrolein		1.50 E+00	1.10 E+04						2.18 E+03	1.10 E+04			1.99 E+00
Aldehydes (unspecified)													
Ammonia (NH3)	9.55 E+01			1.19 E-01					3.18 E+00	5.90 E-02			
Ammonia Chloride													
Anthanthrene													
Anthracene		3.10 E+00	2.30 E+01						3.62 E-02	2.02 E-02			
Antimony (Sb) and Antimony Compounds		1.20 E+00	2.20 E-03						1.93 E+04	3.78 E+03			
Aromatic HC (unspecified)													
Arsenic (As) and Arsenic Compounds		2.00 E+02	2.30 E+00				3.32 E+03	8.16 E+02	2.20 E+05	5.15 E+04			
Benzene (C6H6)		3.00 E-06	5.20 E-03				1.00 E+00	8.48 E-01	1.66 E+01	1.41 E+01			2.46 E-01
Benzo(a)anthracene		8.30 E+02	2.70 E+04				1.42 E+01	5.54 E-01					
Benzo(a)pyrene		2.20 E+02	5.30 E+04				1.62 E+03	1.07 E+01					
Benzo(b,j,k)fluroanthene							6.43 E+01	4.71 E+02					
Benzo(e)pyrene													
Benzo(g,h,i) perylene													
Benzyl Chloride		3.80 E-03	4.10 E+00				4.79 E-01	7.43 E-02	2.34 E+01	3.62 E+00			

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
Beryllium (Be)		1.70 E+02	3.30 E+01				2.78 E+01	7.73 E-47	4.01 E+05	1.41 E+03			
Biphenyl									6.71 E-01	8.55 E+00			
Bis(2-Ethylhexyl) Phthalate (DEHP)		1.60 E-01	4.40 E-02				2.42 E-01	1.00 E-01	4.67 E+01	1.97 E+01			
Bromoform							1.95 E+00	2.00 E+00	7.06 E+02	7.18 E+02			
Cadmium (Cd) and Cadmium Compounds		1.40 E+03	4.60 E+01				8.34 E+01	3.90 E-49	4.95 E+06	3.59 E+05			
Carbon Dioxide (CO2)						1.00 E+00							
Carbon Disulfide (CS2)		1.10 E-02	1.10 E+01						4.69 E+00	6.25 E+00			
Carbon Monoxide (CO)													1.66 E-02
Carbon Tetrachloride		3.30 E-03	1.50 E-02				8.22 E+02	7.81 E+02	1.38 E+04	1.32 E+04		1.10 E+00	
CFC 12									2.69 E+01	2.14 E+01		1.00 E+00	
CFC/HCFC/HFC not specified elsewhere												1.00 E+00	
Chlorine (Cl2)													
Chlorobenzene									2.86 E+00	1.41 E+01			1.95 E-01
Chloroform		1.20 E-04	1.50 E-02				4.57 E+00	4.18 E+00	7.99 E+01	7.94 E+01			
Chromium (Cr) and Chromium Compounds		4.50 E+01	5.20 E-01				1.67 E+02	4.04 E-46	6.36 E+03	6.89 E+02			
Chrysene		6.00 E+00	4.50 E+00				2.04 E+00	9.88 E-01					
Cobalt (Co)		8.20 E+01	5.90 E+00						7.90 E+04	2.46 E-43			
Copper (Cu) and Copper Compounds		1.80 E+02	5.00 E+01						2.78 E+04	1.73 E+04			
Coronene													
Cumene									3.12 E-01	8.59 E-01			
Cyanide													
Dichloromethane (CH2Cl2)		1.80 E-05	4.10 E-02				6.15 E-01	3.68 E-01	4.36 E+01	2.54 E+01			2.38 E-02
Dimethyl Sulfate							1.41 E+02	1.97 E-01					
Dioxins (unspecified)		3.10 E+06	9.50 E+06				1.81 E+09	1.02 E+09	2.29 E+12	1.28 E+12			
Ethyl Chloride/Chloroethane									2.04 E-01	2.04 E-01			
Ethylbenzene (C8H10)		4.40 E-06	5.30 E-01						3.32 E-01	7.97 E-01			7.32 E-01
Ethylene (C2H4)		7.70 E-05	1.60 E-04										2.46 E+00
Ethylene Dibromide							8.43 E+00	1.59 E+01	4.08 E+03	3.51 E+03			
Ethylene Dichloride		5.50 E-05	8.20 E-04				5.33 E+00	5.79 E+00	1.89 E+01	2.03 E+01			
Fluoranthene		5.80 E+01	3.30 E+02						2.12 E+01	1.96 E+01			

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
Fluorides													
Fluorine (F2)									3.40 E+00	4.13 E+01			
Formaldehyde		2.90 E-02	4.50 E+01				3.02 E-03	3.00 E-04	4.77 E+00	5.21 E-01			2.25 E+00
Furans (unspecified)									3.41 E+01	7.90 E+01			3.54 E+00
Hexane									6.07 E-01	1.75 E+01			4.16 E-01
Hexavalent Chromium		3.80 E+02	9.90 E+00										
Hydrocarbons (unspecified)													
Hydrochloric Acid (HCl)	4.47 E+01								3.88 E-01	1.88 E+04			
Hydrofluoric Adic (HF)	8.13 E+01												
Hydrogen (H2)													
Indeno(1,2,3-cd)pyrene		6.60 E+00	4.40 E+03				8.20 E+01	7.26 E+03					
Isophorone (C9H14O)							1.32 E-04	3.11 E-03	8.14 E-03	3.94 E-01			
Kerosene													
Lead (Pb) and Lead Compounds		2.40 E+01	1.30 E-01				3.55 E+01	2.56 E+00	1.50 E+06	1.09 E+05			
Magnesium (Mg)													
Manganese (Mn)									7.94 E+03	9.15 E+00			
Mercaptan													
Mercury (Hg) and Mercury Compounds		2.40 E+05	1.20 E+02						1.93 E+07	1.89 E+07			
Metals (unspecified)													
Methane (CH4)						2.30 E+01							3.68 E-03
Methyl Bromide		5.50 E-02	3.80 E-02						1.23 E+04	6.82 E+03		6.00 E-01	5.96 E-03
Methyl Chloride							3.52 E+00	1.97 E+00	5.39 E+02	3.00 E+02			
Methyl Ethyl Ketone									1.14 E-01	3.31 E-02			
Methyl Hydrazine							3.27 E-01	3.18 E+00					
Methyl Methacrylate									1.39 E-01	1.90 E+00			
Methyl Tert Butyl Ether (MTBE)									9.02 E-02	3.53 E-01			3.30 E-01
Naphthalene		3.80 E-02	2.80 E+02						1.27 E+01	3.40 E+01			7.52 E-01
Nickel (Ni) and Nickel Compounds		1.30 E+02	3.20 E+00				3.61 E+00	1.14 E-47	8.26 E+03	6.68 E+01			
Nitrogen Dioxide (NO2)	4.00 E+01			4.43 E-02					1.01 E-02	2.32 E-02			1.24 E+00
Nitrogen Oxides (NOX)	4.00 E+01			4.43 E-02							2.21 E-03		1.24 E+00

Emissions	**Acidification (air) H+ Moles eq**	**Ecotoxicity (air) kg 2,4-D eq**	**Ecotoxicity (water) kg 2,4-D eq**	**Eutrophication (air) kg N eq**	**Eutrophication (water) kg N eq**	**Global Warming (air) kg CO2e**	**Human Health Cancer (air) kg benzen eq**	**Human Health Cancer (water) kg benzen eq**	**Human Health Non-cancer (air) kg toluen eq**	**Human Health Non-cancer (water) kg toluen eq**	**Human Health Criteria kg PM2.5 eq**	**Ozone Depletion (air) kg CFC-11 eq**	**Photochemical Smog (air) kg Nox eq**
Nitrous Oxide (N2O)						2.96 E+02							
Non Methane Volatile Organic Compounds (NMVOC)													9.68 E-01
Organics (unspecified)													
Organo-Chlorine (unspecified)													
Particulates (PM10)											8.34 E-02		
Particulates (unspecified)													
Perchloroethylene		6.70 E-04	1.80 E-02				1.82 E+00	1.42 E+00	2.16 E+02	1.55 E+02			2.86 E-02
Perylene													
Phenanthrene		5.10 E-01	1.00 E+01										
Phenols		3.80 E-02	9.30 E-01						5.71 E-02	5.38 E-03			9.15 E-01
Polyaromatic Hydrocarbons (total)													
Propionaldehyde													
Propylene									7.07 E-03	7.43 E-02			3.07 E+00
Propylene Oxide		7.20 E-03	6.50 E-01				4.53 E-01	5.94 E-01	1.03 E+02	6.08 E+01			1.04 E-01
Pyrene									2.84 E+00	5.93 E-01			
Selenium (Se) and Selenium Compounds		6.50 E+01	1.00 E-01						2.12 E+04	4.16 E+03			
Styrene		1.20 E-06	2.10 E-03						2.88 E-02	7.51 E-01			6.20 E-01
Sulfur Dioxide (SO2)	5.08 E+01								7.42 E-04	1.24 E-03	1.39 E-02		
Sulfur Oxides (SOX)	5.08 E+01												
Toluene (C7H8)		4.50 E-06	9.70 E-03						1.00 E+00	1.33 E+00			1.03 E+00
Trichloroethane		8.90 E-04	7.10 E-03						2.00 E+02	1.88 E+02		1.00 E-01	2.21 E-02
Trichloroethylene		4.00 E-06	2.40 E-03				5.86 E-02	1.89 E-01	9.82 E-01	2.64 E+01			2.54 E-04
Vinyl Acetate									1.93 E+00	1.33 E+00			
Volatile Organic Compounds (VOC) (unspecified)													9.68 E-01
Xylenes (C8H10)									2.34 E-01	5.50 E-01			
Zinc (Zn) and Zinc Compounds		3.80 E+01	9.20 E-01						4.99 E+02	3.56 E+01			
Water Emissions													
1-Methylfluorene													
2,4 Dimethylphenol									3.34 E-01	2.22 E+00			
2-Hexanone													9.16 E-01

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
2-Methyl Naphthalene													
4-Methy-2-Pentanone									7.33 E-01	6.17 E-01			1.15 E+00
Acetone									3.61 E-01	2.27 E-01			1.19 E-01
Acids (unspecified)													
Alkylated Benzenes													
Alkylated Fluorenes													
Alkylated Naphthalenes													
Alkylated Phenanthrenes													
Aluminum (Al) and Aluminum Compounds									3.04 E+04	2.42 E+01			
Ammonia (NH3)	9.55 E+01			1.19 E-01					3.18 E+00	5.90 E-02			
Ammonia/Nitrogen	9.55 E+01			1.19 E-01					3.18 E+00	5.90 E-02			
Ammonium					7.79 E-01								
Antimony (Sb) and Antimony Compounds		1.20 E+00	2.20 E-03						1.93 E+04	3.78 E+03			
Arsenic (As) and Arsenic Compounds		2.00 E+02	2.30 E+00				3.32 E+03	8.16 E+02	2.20 E+05	5.15 E+04			
Barium (Ba) and Barium Compounds		9.00 E+00	8.50 E-04						9.62 E+02	1.26 E+02			
Benzene (C6H6)		3.00 E-06	5.20 E-03				1.00 E+00	8.48 E-01	1.66 E+01	1.41 E+01			2.46 E-01
Benzoic Acid									2.81 E-02	5.26 E-03			
Beryllium (Be)		1.70 E+02	3.30 E+01				2.78 E+01	7.73 E-47	4.01 E+05	1.41 E+03			
Biological Oxygen Demand (BOD)					5.00 E-02								
Biphenyl									6.71 E-01	8.55 E+00			
Boron (B)													
Bromide													
Cadmium (Cd) and Cadmium Compounds		1.40 E+03	4.60 E+01				8.34 E+01	3.90 E-49	4.95 E+06	3.59 E+05			
Calcium (Ca) and Calcium Compounds													
Carbon Disulfide (CS2)		1.10 E-02	1.10 E+01						4.69 E+00	6.25 E+00			
Carbon Trioxides (CO3-)													
Chemical Oxygen Demand (COD)					5.00 E-02	5.00 E-02							
Chlorides (unspecified)													
Chlorine (Cl2)													
Chromium (Cr) and Chromium Compounds		4.50 E+01	5.20 E-01				1.67 E+02	4.04 E-46	6.36 E+03	6.89 E+02			

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
Cobalt (Co)		8.20 E+01	5.90 E+00						7.90 E+04	2.46 E-43			
Copper (Cu) and Copper Compounds		1.80 E+02	5.00 E+01						2.78 E+04	1.73 E+04			
Cresols									5.45 E+00	1.17 E-01			
Cyanide													
Cymene													
Detergents/Oils													
Dibenzofuran													
Dibenzothiophene													
Dibenzo-thiophenes													
Dissolved Organics (Non Hydrocarbon)													
Dissolved Solids (unspecified)													
Ethylbenzene (C8H10)		4.40 E-06	5.30 E-01						3.32 E-01	7.97 E-01			7.32 E-01
Fluorine (F2)									3.40 E+00	4.13 E+01			
Hardness													
Hexanoic Acid													
Hydrocarbons (unspecified)													
Iron (Fe) and Iron Compounds													
Lead (Pb) and Lead Compounds		2.40 E+01	1.30 E-01				3.55 E+01	2.56 E+00	1.50 E+06	1.09 E+05			
Lithium (Li)													
Magnesium (Mg)													
Manganese (Mn) and Manganese Compounds									7.94 E+03	9.15 E+00			
Mercury (Hg) and Mercury Compounds		2.40 E+05	1.20 E+02						1.93 E+07	1.89 E+07			
Metals (unspecified)													
Methyl Chloride							3.52 E+00	1.97 E+00	5.39 E+02	3.00 E+02			
Methyl Ethyl Ketone									1.14 E-01	3.31 E-02			
Molybdenum (Mo)		6.90 E+00	1.50 E-01						3.25 E+04	9.37 E+03			
Naphthalene		3.80 E-02	2.80 E+02						1.27 E+01	3.40 E+01			7.52 E-01
Nickel (Ni) and Nickel Compounds		1.30 E+02	3.20 E+00				3.61 E+00	1.14 E-47	8.26 E+03	6.68 E+01			
Nitrates													
Nitrogen (N) (other)					9.86 E-01								
Nitrogen tetrahydride (NH4)													

Emissions	Acidification (air) H+ Moles eq	Ecotoxicity (air) kg 2,4-D eq	Ecotoxicity (water) kg 2,4-D eq	Eutrophication (air) kg N eq	Eutrophication (water) kg N eq	Global Warming (air) kg CO2e	Human Health Cancer (air) kg benzen eq	Human Health Cancer (water) kg benzen eq	Human Health Non-cancer (air) kg toluen eq	Human Health Non-cancer (water) kg toluen eq	Human Health Criteria kg PM2.5 eq	Ozone Depletion (air) kg CFC-11 eq	Photochemical Smog (air) kg Nox eq
Nitrogen Trioxide					9.86 E-01								
Organics (unspecified)													
Pentamethyl Benzene													
Phenanthrene		5.10 E-01	1.00 E+01										
Phenols		3.80 E-02	9.30 E-01						5.71 E-02	5.38 E-03			9.15 E-01
Phosphorous (P) and Phosphorous Compunds				1.12 E+00	7.29 E+00								
Potassium (K) and Potassium Compounds													
Selenium (Se) and Selenium Compounds		6.50 E+01	1.00 E-01						2.12 E+04	4.16 E+03			
Silver (Ag) and Silver Compounds									4.28 E+03	1.19 E+03			
Sodium (Na) and Sodium Compounds													
Strontium (Sr) and Strontium Compounds													
Sulfates (SO4-)							1.41 E+02	1.97 E-01					
Sulfur (S) and Sulfur Compounds													
Sulfuric Acid													
Surfactants													
Suspended Solids													
Thallium (Tl)		2.20 E+02	2.00 E+00						3.23 E+07	7.14 E+06			
Tin (Sn)		6.40 E+00	1.80 E-01						1.02 E+02	6.27 E-02			
Titanium													
Toluene (C7H8)		4.50 E-06	9.70 E-03						1.00 E+00	1.33 E+00			1.03 E+00
Total Alkalinity													
Total Organic Carbon (TOC)													
Vanadium (V)		2.10 E+02	1.60 E+02						3.00 E+03	1.86 E+03			
Xylenes (C8H10)									2.34 E-01	5.50 E-01			
Yttrium (Y)													
Zinc (Zn) and Zinc Compounds		3.80 E+01	9.20 E-01						4.99 E+02	3.56 E+01			