PERFORMANCE BASED DESIGN AND EVALUATION OF REINFORCED CONCRETE BRIDGES

by

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ABSTRACT

Performance based design (PBD) has been deemed as one of the most promising design methods in the past few decades. It reduces the uncertainties underline the tradition force based design (FBD) and acts as an efficient communication tool between technical and non-technical people. Canadian Highway Bridge Design Code (CHBDC) has initiated PBD in Canada in 2014, which brought in one of the biggest changes to the new version of the design code. For Lifeline bridges and irregular Major Route bridges, the code requires PBD to be used to explicitly demonstrate structural performance. As per the code, Regular Major Route bridges can be designed by using either FBD or PBD method. In this study, a multi-bent concrete highway bridge is designed using both FBD and PBD based on CHBDC 2014, and FBD based on CHBDC 2006. The evaluation of different designs is performed to determine which method is more conservative. Soil-structure interaction is incorporated using p-y method in the design and analysis. Dynamic time-history analyses are performed to assess the seismic performance. The assessment is based on the maximum strain limits from CHBDC 2014. The results reveal that the PBD in CHBDC 2014 is highly conservative in comparison with FBD in current and previous design codes. This is because CHBDC 2014 requires rebar yielding shall not happen at 1/475-year earthquake event. Eliminating rebar yielding at 1/475-year event may be very challenging to achieve in high seismic regions and 1/475-year event may dominate other design levels. After performing the PBD, a displacement based design approach is also used to examine the performance criteria from the code. It is shown that by using displacement based approach the PBD could be simplified for regular bridges.
Additionally, a series of charts of column drift versus steel strain are presented to facilitate future engineering designs. At the end, the methodology of the next generation PBD is utilized to compare the seismic performance of bridges in terms of engineering demand parameters and decision variables.
PREFACE

A portion of this study has been submitted to peer-reviewed journals and conferences for publication. All the studies in the following papers have been solely carried out by the author. The thesis supervisor was responsible for the research guidance and review of the work produced by the author.

List of Publications Related to this thesis


Qi Zhang; M. Shahria Alam; Saqib Khan and Jianping Jiang. Seismic performance comparison between force-based and performance-based design of a highway bridge as per CHBDC 2014. Canadian Journal of Civil Engineering (under review).
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LIST OF SYMBOLS

\( c \) distance from neutral axis to compression edge

\( dm \) damage measure

\( dv \) decision variables

\( d \) distance from neutral axis to tension reinforcement

\( D \) column diameter

\( edp \) engineering demand parameter

\( E \) modulus of elasticity of steel

\( f_{ce}' \) expected concrete strength

\( f_{cc} \) confined concrete strength

\( F_m \) maximum design force as per displacement based design

\( H \) column height

\( im \) seismic intensity measure

\( L_{sp} \) length of strain penetration

\( P \) probability of a structure exceeding limit states

\( T \) effective period

\( T_c \) corner period

\( \alpha_1 \) ratio of average stress in a rectangular compression block to the specified concrete strength
$\beta_1$ a concrete compressive stress area factor

$\varnothing_y$ yield curvature

$\varnothing_c$ concrete curvature

$\varnothing_s$ rebar curvature

$\Delta d$ design displacement

$\Delta c$ displacement at the corner period

$\Delta_B$ displacement of bearing

$\Delta y$ yield displacement

$\Delta p$ is the plastic displacement of pier

$\Delta m$ maximum displacement

$\varepsilon_y$ yield strain

$\varepsilon_c$ concrete strain

$\varepsilon_s$ steel strain

$\theta$ drift

$\zeta$ equivalent viscous damping

$\zeta_e$ elastic damping

$\zeta_h$ hysteretic damping

$\rho_l$ reinforcement ratio
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Dedicated to my parents

Mr. Wenge Zhang

And

Mrs. Lili Jiang
Chapter 1: INTRODUCTION AND THESIS ORGANIZATION

1.1 Introduction

Performance based design (PBD) was initiated in 1980s in the United States (Hamburger et al., 2004). Engineers have been practicing PBD for several decades since then. The fundamental methodology of PBD is working in terms of ends rather than means (Gibson, 1982). The PBD is performed based on clear, specific, and measurable design criteria, such as material strains and drift ratios. The specific structural design can be based on any reasonable design methods, as long as the seismic performance can be demonstrated explicitly in terms of serviceability and damages.

The current PBD mainly has two advantages over the tradition force based design (FBD). Firstly, PBD reduces the uncertainties in the FBD and demonstrate performance using measurable criteria. Force is not measurable and can only be proved by its effects (such as strains and displacements). Therefore, the earthquake force cannot be a specific and clear design criterion. Secondly, the PBD not only considers life safety, it incorporates post-earthquake behaviors and reduces economic losses. The PBD starts from the seismic fragility analysis. Based on the bridge fragility, direct structural damage and direct losses can be estimated. From the bridge performance and transportation data, the indirect losses can be estimated. In the current design codes, transportation data and indirect losses are represented by bridge importance categories, for example Lifeline Bridge, Major Route Bridge and Other Bridges in the Canadian Highway Bridge Design Code (CHBDC) (CSA, 2014). In the CHBDC (CSA, 2014), PBD can be used for all design cases while
FBD is only permitted for certain cases. Regular Major-Route bridges can be designed by using FBD or PBD method. Major-Route Bridge is the most common category for highway bridges and is the focus of this study.

In current practices, PBD is achieved by performing multiple hazard level designs that meet the requirements at different service levels. In Canada there are many bridges designed using the PBD methods, such as the recent constructed Port Mann Bridge, Vancouver (Jones et al., 2013) and the Vancouver Evergreen Line Rapid Transit Project (Khan & Jiang, 2015). In the recent published CHBDC (CSA, 2014), PBD has been first time written in the bridge code in Canada. For Major Route Bridges, PBD requires the bridge to be limited in minimal damage, repairable damage and extensive damage in the event of 1 in 475, 975 and 2475 years event. It is expected that the performance of a bridge is consistent or similar either it is designed by FBD or PBD approach as per the coded. However, to date no information is available how the PBD is calibrated to the FBD. This leads to the necessity of comparing FBD with PBD as per the CHBDC (CSA, 2014).

1.2 Objective of the study

The objective of this study is to compare the seismic performance of a multi-column bent reinforced concrete highway bridge designed as per old (CSA S6-06) and new Canadian Highway Bridge Design Code (CSA S6-14). Furthermore, bridge designs are also compared based on the next generation of PBD criteria, which incorporates repair cost and repair time. In order to fulfill the objectives, this study performs:
1. Seismic behavior comparison between bridges designed using force based and PBD method.

2. Analysis of the performance criteria in the code based on displacement based design approach.


1.3 Scope of this research

In order to achieve the objectives of this research, a literature review on performance based design is conducted firstly. Review on displacement based design (DBD) is also performed since it is one of the most promising design methods. FBD and PBD are compared by performing a multi-column concrete bridge design case study. The soil structure interaction is investigated in both design and analysis phases. After performing the different designs, the seismic performance is evaluated and compared. Furthermore, the performance criteria in Canadian Highway Bridge Design Code (CSA, 2014) are evaluated using displacement based approach. At the end of this study, the framework of the next generation PBD is followed to compare three bridge designs in terms of repair cost and repair time.

1.4 Thesis organization

The thesis has six chapters where the present chapter presents introduction, objectives and scope of the research.
In Chapter 2, an overview of the current state-of-the-art of performance based design is performed. The initiation, current practices, and future of performance are discussed. The chapter presents the fundamentals of performance based design. The philosophy of performance based approach and the necessity of performance based design are explored.

In Chapter 3, a four span concrete bent highway bridge is designed using FBD and PBD methods as per CHBDC 2014, and FBD method as per CHBDC 2006. Soil-structure interaction is considered in the design and analysis. Pushover analyses are carried out on the selected bents to evaluate their seismic performance. Nonlinear dynamic time-history analyses are conducted to rigorously assess the seismic performance of these three designs. Maximum strains of concrete and steel are the output of nonlinear analysis and are used for seismic performance assessment. The three designs are compared in terms of damage states.

Chapter 4 starts with an introduction of displacement-based design (DBD). In this chapter, the methodology of displacement-based design as per CHBDC 2014 is presented. CHBDC (CSA, 2014) initials PBD in Canada with design criteria mainly based on material strains. The material strain limits can be used to calculate design displacements following the DBD method. Additionally, this study also presents the relations between steel strain and column drift for varying column size and reinforcement ratio. The charts can be used to accelerate engineering design process.
Chapter 5 presents the bridge evaluation methodology based on the next generation of PBD. The next generation of PBD not only explicitly demonstrates structural serviceability but also predicts economic losses and repair time explicitly. This chapter compares three designs of two span concrete bridge in terms of structural damage, repair cost and repair time. Fragility analysis using cloud method is performed and the damage of bridge components is used to calculate repair cost and repair time.

Finally, conclusions of this research and recommendations for future research are presented in Chapter 6.
Chapter 2: OVERVIEW OF THE CURRENT STATE-OF-THE-ART OF PERFORMANCE BASED DESIGN

2.1 General

Bridges are essential components for transportation systems. The damage of bridges not only affects its immediate users but also brings serious aftermath to earthquake events. Without functional bridges it would be extremely difficult to provide aid to destroyed areas. Traditionally, bridges were designed by force base design (FBD) approaches. However, the transition from FBD to performance based design (PBD) is happening quickly since many design guidelines have adopted PBD methodology (CSA, 2014; FHWA-NHI, 2014). PBD is based on probabilistic demand and capacity models which will lead to a better understanding of risk control and management (Mackie & Stojadinović, 2005). PBD in the earthquake engineering field was initiated in the United States in 1980s (Hamburger et al., 2004). Since then, engineers have been practicing PBD for several decades (Hamburger et al., 2004). The fundamental methodology of PBD is based on the end performance rather than the design approaches (Gibson, 1982). PBD is conducted based on clear, specific, and measurable design criteria, such as material strains and drift ratios. The structural design can be based on any reasonable design methods, as long as the seismic performance is explicitly demonstrated in terms of serviceability and damages. The design criteria can be based on strains or drift limits from the code, or the project agreement between owners and designers.
The most widely accepted FBD method is Limit States Method. It can use the plastic range of capacities and incorporate load factors for the demands (Allen, 1975; Priestley et al., 2007a). However, the FBD cannot explicitly connect structure performance with design processes. PBD mainly has two advantages over the traditional force based design (FBD). Firstly, PBD reduces the uncertainties in the FBD since PBD demonstrates performance using measurable criteria such as strain and drift. The criteria can always be demonstrated and checked explicitly. Secondly, the PBD not only considers life safety issues, it also incorporates post-earthquake behaviors and the mitigation of economic losses. PBD is necessary for seismic engineering because the traditional FBD has a number of shortcomings: (1) FBD cannot properly predict the effective stiffness of structures; (2) FBD cannot properly distribute the base shear to various columns after yielding; and (3) FBD cannot accurately consider soil foundation interactions and dual load path (Priestley et al., 2007a). In the past, it was reported that FBD frequently failed to limit structural damages in major earthquakes (Ghobarah, 2001).

Figure 2.1 shows a brief illustration of PBD framework. PBD incorporates not only the seismic demands, but also the social demands. PBD usually starts from the seismic fragility analysis. Then based on the bridge fragility, direct structural damages such as concrete spalling, bearing failure and corresponding losses will be estimated. From the bridge performance and transportation data, the indirect losses caused by traffic delay and such will be predicted at the end. In the current design code, indirect losses are represented by bridge importance. As per CHBDC (CSA, 2014), Lifeline bridges are large and unique structures that are vital to transportation system and are time-consuming
to repair or replace. Major Route Bridges are the key routes of the regional transportation network that facilitate post-earthquake emergency response, security and defense purposes. Bridges which does not belong to the above two categories are defined as Other Bridges.

![PBD framework](image)

Figure 2.1 PBD framework

In Canada, there are several bridge projects delivered using the PBD method, such as the recent constructed Port Mann Bridge, Vancouver (Jones et al., 2013) and the Vancouver Evergreen Line Rapid Transit Project (Khan & Jiang, 2015). In the recent published Canadian Highway Bridge Design Code (CSA, 2014). PBD requires the Major Route Bridges to be limited in minimal damage, repairable damage and extensive damage in the event of 1/475, 1/975 and 1/2475 years events, respectively (CSA, 2014). At the same time, traditional FBD is also permitted. It is expected that the performance of a bridge is
consistent in terms of serviceability after earthquake events regardless of the design approach (FBD or PBD) as per the code.

2.2 The development of performance based design

The initial development of PBD dates back to 1980s (Hamburger et al., 2004). The major reason for the initiation was that building owners would not invest a considerable amount of money in building retrofits unless they know what corresponding results would happen after earthquakes (Hamburger et al., 2004). Building and bridge owners have to know what will happen if the structure is not retrofitted, or if the structure is retrofitted with an acceptable cost before investing money. In this sense, the function of PBD is being a communication tool between engineers and non-technical decision makers. The field of structural design of bridges is similar to buildings on both technical and non-technical sides. On the technical side, the task of engineer is to deliver structures that can protect life safety and reduce losses. On the non-technical side, the common goal of different sectors is to optimize and make the best fit decisions.

The term “performance-based design” seems to only be used for civil structure designs. However, numerous industries have adopted the methodology of PBD. For example, in automotive industry, many of designers have to perform crash test or analyses before delivering their product to the market (Dugoff, 1970). The designers also have to predict how the product would help protect life safeties. It is a part of the quality control (QC) process in many industries to explicitly demonstrate the performance of the product. Based on the target market and customers, the products are designed in various ways. The
reason why the performance base design of structures was initiated later than many other industries may be that infrastructure design has several unique characteristics compared to other industries. Firstly, structural design is highly dependent on geographical conditions. Different regions usually have very different hazard demands and soil conditions, which determine different level of engineering demands. Secondly, the transportation demands, which also affect the designs, are different for different projects. For example, bridge design has to be customized to the type of vehicles that will use it. Thirdly, the natural environment and the material availability are different from area to area and as a result, the structure type varies. The above mentioned three characteristics make each bridge design and construction project different from each other and make it difficult for engineers to predict its performance. Additionally, what makes the bridge projects different from other industries is that the bridge projects are usually in large scale and expensive. Therefore, before constructing and testing a bridge, it is difficult and expensive to predict the end performance compared to what is performed in other industries. However, history has shown that the society cannot afford to build structures that are vulnerable to earthquakes. In 2015, the Nepal earthquake caused more than 8,500 deaths and economic loss in the order of 10 billion dollars. The economic loss is about 50% of the gross domestic product (GDP) of the country (Goda et al., 2015).

The efforts of predicting the performance of bridges have been gone through several decades. ATC 13 (ATC, 1985) is one of the earliest reports proposing PBD. It focuses on earthquake damage repair evaluation and repair cost in California area. This report includes the methods and data for loss estimates in limited areas. It is a guideline for
estimating economic losses, repair time, deaths, and injuries. This document is also used by professionals to estimate probable maximum loss (PML) for insurance and other decision makings. ATC 14 provides methodology for building evaluations under seismic hazards (Miranda & Bertero, 1994). Following that, FEMA-356 was published by FEMA (2000a) providing seismic rehabilitation guidelines. As per FEMA-356, most of the building that are rehabilitated would meet the performance level at the design level earthquakes. The above mentioned documents were mainly developed for buildings. Performance based design of bridges was extensively researched by Mackie & Stojadinović, (2007), Priestley (2000) and other researchers (Billah & Alam, 2014c; Dawood & ElGawady, 2013). A comprehensive study from U.S. National Cooperative Highway Research Program (2013) presented a summary and guideline for performance based bridge seismic design. These documents are the basis of the first generation PBD. The first generation PBD is mainly based on structural responses such as drifts, material strains, and ductility demands. Although the PBD was initiated from structural design of buildings, the relative research is applicable to bridge structures. A timeline of the major development of PBD is presented in Figure 2.2.
Hamburger et al. (2004) concluded that there are still many shortcomings in the current first generation of PBD. Firstly, in the current design approaches, the global behavior is dominant by the weakest components. In many cases, this would result in unnecessarily conservative designs. Secondly, the damage to non-structural components and systems cannot not be properly estimated, whereas, the non-structural damage may cause much higher losses than the damage of structural components. Thirdly, it is unknown if the desired seismic performance can be achieved just by following the coded procedures. For example, the code may states that the force reduction factor for multiple bent reinforced concrete bridge is five regardless of other parameters. However, by taking this assumption the bridge design may not be conservative since shorter columns may not be able to achieve that ductility level. Sheikh and Légeron (2014) revealed that the design procedure from CHBDC 2006 (CSA, 2006) does not automatically achieve the desired performance goals stated by the code. Additionally, the current PBD cannot properly
incorporate future repair cost and repair time, which are the biggest concerns of stakeholders. The above mentioned issues are expected to be addressed in the next generation PBD. In a 10-year project ATC 58 (ATC 2012) started since 2001, a performance based methodology for the seismic assessment considering uncertainties was developed (Hamburger et al., 2004). This project determines the preferred way of communication between engineers and other stakeholders. This project also establishes PBD and assessment methodology.

To date there are numerous publications focused on the study of PBD. The statistics of publications in the past 25 years related to performance based seismic design is shown in Figure 2.3. Since early 1990s, there are at least 5,041 papers investigating performance based design including buildings, bridges and other structures. Among the publications, 864 of them are focused on performance based bridge designs, which represent 17% of publications in all fields. The number of publication increases stably and shows a grown interest in this field. Among publications in all fields, 60% of them are journal papers and 40% are conference papers. For the publication in the field of bridges, 59% are journal papers and 41% are conference papers.
2.3 **Performance based design framework**

Performance based design is an approach that designs structures in terms of end performance (Spekkink, 2005). It explicitly relates structural performance with design process by eliminating intrinsic uncertainties (Arnold et al., 2004; Khan & Jiang, 2015; Lehman & Moehle, 2000). In most cases, nonlinear analyses are required in PBD to demonstrate the target performance. With the fast development in computing technologies, nonlinear analysis can be achieved in more efficient ways with affordable computational resources such as parallel and cloud computing (Bebamzadeh et al., 2014).

PBD is a philosophy that combines engineering and social sciences. Bridge design usually involves multiple alternatives and multiple criteria from experts in different spheres (Billah & Alam, 2014b) rather than only technical perspective. PBD requires engineers with good understanding of nonlinear behavior of structures, soil foundation interactions, and also higher management skills compared to traditional designs. It is not
only being implemented to new bridges constructions, but also to old bridge retrofits (Buckle et al., 2006). Additionally, PBD methodology has been adopted by many researchers in developing new structural systems, such as reinforced concrete frame confined by fiber-reinforced plastic (FRP) (Zou et al., 2007) and shape memory alloy (SMA) reinforced concrete (Billah & Alam, 2015).

The goal of PBD is to protect life safety and reduce regional economic losses. Performance based design is not a simply conservative design compared to FBD. It accounts for the damage of both structural and non-structural members (Filiatrault & Sullivan, 2015). Earthquake loss models of a region are discussed in many research projects and are expected to reduce future losses from earthquake events (Hazus, 1997; Lu et al., 2014; Moore et al., 2002). An excellent PBD framework that consistently considers overall design issues was developed by Moehle and Deierlein (2004). This framework is beyond the traditional scope of structural engineering. It not only involves structural seismic demand and damage, but also tracks social demand and losses. The methodology is of great value for stakeholders to make decisions and for code committee to calibrate design provisions. By using PBD, the decision makers will be able to distribute resources based on the social demands (Marsh, 2013). A PBD method considering loss limit was presented by Mackie and Stojadinović (2007), which incorporates more considerations than only force and displacement demands. In PEER’s performance based framework, an overall probabilistic model for earthquake demand is integrated from several sub-models that consider lower level of probabilities. This model is presented in Equation 2.1 (Mackie & Stojadinović, 2005).
\[ P(D > d\nu) = \]

\[
\int_{dm} \int_{edp} \int_{im} G_{DV/DM}(d\nu|dm) \cdot \left| dG_{DM/EDP}(dm|edp) \right| \cdot \left| dG_{EDP/IM}(edp|im) \right| \cdot \frac{1}{dG_{IM}(im)} \]

(2.1)

where, \( P \) is the total probability of a structure exceeding limit states, \( DM \) is damage measure, \( EDP \) is engineering demand parameter, \( IM \) is seismic intensity measure, \( DV \) is decision variables.

One method of presenting PBD results is fragility curves, which shows the probabilities of exceeding design criteria at various hazard levels (Mackie & Stojadinović, 2005). Numerous researchers have used fragility curves to describe the seismic behavior of structures (Choi et al., 2004; Hwang et al., 2001; Kim & Feng, 2003; Billah & Alam, 2015; Padgett & DesRoches, 2009; Shinozuka et al., 2000). The first generation of fragility curves were mostly based on empirical studies, especially based on the damages in 1994 Northridge and 1995 Hyogo-ken Nanbu earthquakes (Gardoni et al., 2002; Mackie & Stojadinović, 2005). However, the data from past experience is very limited and as a result, it will be difficult to predict future structural fragilities based on empirical method. Therefore, the second generation of fragility analysis is largely based on analytical studies. The analytical studies can provide more insight on existing bridge structures and new designs that have not experienced any earthquakes, such as concrete bridge pier reinforced with super-elastic shape memory alloy (Billah & Alam, 2014).
Many efforts have been done to relate seismic performance to structural serviceability 
(Lu et al., 2011). The relation between the bridge residual load carrying capacity and the
residual displacement was researched by Mackie and Stojadinović (2004). In a later
investigation, Mackie et al. (2008b) studied post-earthquake bridge repair cost and time
predictions. Under the same framework, an application named BridgePBEE (Lu et al.,
2011) was developed based on the framework of OpenSees (Mazzoni et al., 2006).
BridgePBEE not only performs nonlinear structural analysis, but also tracks triggered
repair costs after earthquake damage. Forcellini et al. (2012) presented a performance
based assessment example using the program BridgePEBB. The performance criteria
include structural damages and loss levels. It will be of great value for bridge owners to
make decisions since most of social decision makings are based on economic issues.

2.4 Performance based design approaches

Performance based design is not a single specific design method but a framework for
optimized designs. Under the PBD framework, structures can be designed based on any
method including DBD, FBD and other design methods such as energy-based design
(Leelataviwat et al., 2002). Among different design methods, DBD may be the most
prominent method under the PBD framework since it allows engineers to directly control
defformation and thus damages (Dwairi & Kowalsky, 2006).

Displacement based design (DBD) method for bridge was proposed and improved by a
number of researchers (Chopra & Goel, 2001; Fajfar, 1999; Kowalsky, 2002; Moehle,
1992; Priestley et al., 2007a). It should be noted that although DBD is one of the most
promising methods, it still has many potential limitations when applied to long span bridges and irregular bridges (Ayala et al., 2007). This is because DBD is usually restricted to structures for which their deformed shape is easily estimated (Sullivan et al., 2003). Additionally, the shear design is not properly addressed (Reza et al. 2014, Priestley et al., 2007b). Many innovative approaches were developed by researchers to expend the application of displacement based design. DBD was adopted by researcher for various types of bridges such as continuous concrete bridges (Kowalsky, 2002), RC arch bridges (Khan et al., 2013), long span bridges (Adhikari et al., 2010), isolation systems (Fardis, 2007; Sanchez-Flores & Igarashi), precast pre-stressed wall systems (Rahman & Sritharan, 2011), deck integrated piers (Bardakis & Fardis, 2011), irregular bridges (Catacoli et al. 2012; Kappos et al., 2013) and hybrid sliding-rocking bridge (Madhusudhanan & Sideris, 2015). P-Δ effects in displacement based design was investigated and addressed by Suarez and Kowalsky (2011) and Wei et al. (2011). The foundation flexibility issue for displacement based design was investigated by Paolucci et al. (2013). A uniform seismic risk design with displacement based design approach was proposed by Wang et al. (2014). The influence on DBD from soil structure interaction was investigated by many researchers (Calvi et al., 2014; Ni et al., 2014). Displacement based design is being used for both bridge designs and bridge assessments. A number of case studies were conducted by Cardone et al. (2011). The authors derived nine bridge configurations by changing a reinforced concrete bridge layout from an Italian highway project. The prototype of these bridges was a multi-span simply-supported bridge. The studied layouts included the pier heights, bearing types and deck types. The authors estimated damage states corresponding to various peak ground acceleration values. The
design displacement was calculated by using Iterative Eigenvalue analysis method. It was confirmed the displacement based design procedure has a good accuracy based on the non-linear time history analyses. In a later research, Cardone (2012) presented two different methods for Direct Displacement-Based seismic assessment by using Displacement Adaptive Pushover (DAP) analysis and Iterative Eigenvalue Analysis (IEA). These two methods were used to conduct a case study on the Greek Egnatia Motorway Bridge. The authors concluded that the results of two methods were consistent and the average difference was less than 5%. Both the two methods provide enough accuracy for designs.

Şadan et al. (2013) presented displacement based design examples of multi-span reinforced concrete bridges with single piers. Detailed displacement based design assessment procedures and the limitations were discussed. There are still some limitations because this method cannot take girder, bearing and shear key failures into account. In this study Effective Mode Shape method (EMS) was used to capture the high mode of the structures (beyond the first two modes). The authors also assessed the bridges by using response spectrum analysis (RSA), which is a classic force-based assessment method. The results from displacement based design and RSA were compared using incremental dynamic analysis. Results showed that the accuracy of displacement based design is better than RSA in their cases.

In New Zealand, a number of existing bridges were assessed using displacement based method. Most of these bridges were designed before 1975, when the standards were
much lower than current requirements (Novakov et al., 2009). The authors performed case studies of displacement based seismic assessment of bridges. In the case of Little Grey River Bridge, its transverse direction did not have enough strength and ductility. In its longitudinal direction, there were discontinuities when shear failure happened at the abutment anchors. After the retrofit design, the authors compared the results of force based method with those of displacement based method. Results revealed that force based method overestimates displacement demand. Meanwhile, in the case of Boundary Stream Bridge, the investigators checked the possibility of unseating of the girder at the abutment. The force based method predicted lower displacement than displacement based design, thus displacement based design is in the conservative side in the latter case (Boundary Stream Bridge). Ni et al. (2014) performed assessments for multi-span reinforced concrete bridge by considering soil-structure interaction (SSI). The results predicted by displacement based design method were compared with the results from dynamic finite element analysis. The authors concluded that the displacement based design can provide reliable results and be of great help when doing screening on a large amount of bridges. It was also mentioned that SSI effects can reduce the seismic demand on structures based on their analyses.

### 2.5 Performance based design criteria

In PBD, the design criteria such as strains are selected by designers and assigned to different earthquake events. Structures have to be designed for multiple criteria which mainly include safety, serviceability and economy (Bertero, 1996). There are a number of guidelines (PEER, 2010) and codes (CSA, 2014) that have requirements on multiple level
seismic designs. The design criteria in CHBDC 2014 for major route bridges are shown in Figure 2.4. Usually the structures are designed for a lower level and checked for a higher level hazard. At the lower level, the structure should remain functional (serviceability) and at the higher level the structure should protect the life safety.

![Figure 2.4 Performance criteria (CHBDC 2014)](image)

The different levels of earthquake are defined by their return periods. The risk analysis is based on Poisson model (Wang, 2006). In this model, the probability of n earthquakes occur during an time period (t years) is

$$P(n, t, \tau) = \frac{e^{-\frac{t}{\tau}}(\frac{t}{\tau})^n}{n!} \tag{2.2}$$

where, $\tau$ is the average recurrence interval of earthquakes equal to or greater than a specific magnitude. The probability of the design earthquake happening at least once is
\[ P(n \geq 1, t, \tau) = 1 - e^{-\frac{t}{\tau}} \]  

Different guidelines and codes use different design criteria. In the PEER’s Guidelines for Performance-Based Seismic Design of Tall Buildings (PEER, 2010), it is required that the structure remain essentially elastic with limited damages under an event with a 43-year return period (50% exceedance probability in 30 years). In the recommendation from International Federation for Structural Concrete (fib), the lower level design (Operational) is a 70% probability of being exceeded in the service life (Fardis, 2013). In CHBDC 2014, the lower level design has a 10% probability in 50 years. At this event, major route bridges shall remain elastic. In FEMA-350, Maximum Considered Earthquake (MCE) ground shaking has a 2% probability of exceedance in 50 years (2475 year return period). At the Design Earthquake (DE) ground shaking level, the spectrum is defined as 2/3 of the MCE spectrum (Venture et al., 2000). In FEMA 356 (FEMA, 2000b), which was developed for seismic rehabilitation of buildings, at the lower design level, limited yielding is permitted for both concrete frames and steel moment frames. In the Bridge Engineering Handbook (Wai-Fah & Lian, 2000), First Edition, it was suggested that a return period of 72 ~ 250 year be used for lower level design. In the Second Edition of the Bridge Engineering Handbook (Chen & Duan, 2014), the authors mentioned that the practice in California is that hazard levels are developed in consultation with Seismic Safety Peer Review Panel at the lower design level. In Eurocode 8 Part 2—Seismic Design of Bridges (EC8-2), single design hazard is used and the design level can be specified by the using country. Generally 475 years return period is used with Eurocode (Marsh & Stringer, 2013).
It was noted that AASHTO (2012) and its reference manual from U.S. Federal Highway Administration (FHWA-NHI, 2014) do not specify specific design criteria in terms of strain and drift like CHBDC 2014 (CSA, 2014). The same philosophy is also adopted in Japanese Specification for Highway Bridges (JRA 2002), New Zealand earthquake design standard (NZ-Standard 2004), New Zealand bridge manual (NZ-Transport, 2014) and Chinese Specifications of Earthquake Resistant Design for Highway Engineering (China-MOC, 2008), where only general performance is required rather than specific strain and drift values (Chen & Duan, 2013).

The performance criteria between different codes and guidelines are not consistent. Among all the guidelines and codes, CHBDC 2014 has the strictest criteria at the lower design level. A list of performance criteria at the lower design level from different bridge code is presented in Table 2.1. The design level and criteria are listed for common importance structures.

<table>
<thead>
<tr>
<th>Codes / guidelines</th>
<th>Design level</th>
<th>Design criteria</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Zealand earthquake design standard</td>
<td>1000 years return period with adjustment factors</td>
<td>Cater for all design-level actions</td>
<td>(NZ-Transport, 2014)</td>
</tr>
<tr>
<td>Chinese Specifications of Earthquake Resistant Design for Highway Engineering</td>
<td>70-100 years return period</td>
<td>Remain usable without damage or repair</td>
<td>(China-MOC, 2008)</td>
</tr>
<tr>
<td>AASHTO</td>
<td>1000 years return period</td>
<td>Remain open to all traffic</td>
<td>(AASHTO, 2012)</td>
</tr>
<tr>
<td>CHBDC 2014</td>
<td>475 years return period</td>
<td>No yielding is allowed</td>
<td>(CSA, 2014)</td>
</tr>
</tbody>
</table>
There are a number of bridges designed according to performance based approaches with at least two level hazards. At the lower level design, most of the bridges were design for a return period of no more than 100 years, including Tacoma Narrows Bridge, San Francisco to Oakland Bay Bridge, and Gerald Desmond Bridge (Jones et al., 2013). For the above mentioned bridge designs, the target performance for the bridge is to remain essentially elastic with minor inelastic response. Additionally, Willamette River Transit Bridge and Port Mann Bridge were designed for a 475-year return period at the lower design level, and the target performance was essentially elastic with minor inelastic response (Jones et al., 2013). Carquinez Straits Bridge was designed for a 300-year return period, with minor inelastic response. These design criteria were determined by project members jointly in order to achieve a design that would benefit all the parties.

2.6 Performance-based bridge retrofit

In the United States, about half of the bridges were constructed before 1935 (Golabi & Shepard, 1997). Bridge constructions have been conducted 100 hundred years ago and the deterioration of bridges have altered the new bridge construction to bridge maintenance. The bridge repair cost and repair time are major issues that bridge designers should consider. Bridge repair is an important task since more bridges are closing to the end of their life span. The repair of bridges should firstly guarantee public safety. At the same time, the maintenance should also maximize overall benefit of the whole society, which means that the distribution of funding should take other criteria into consideration.
PBD framework has been developed to guide bridge retrofits. Gordin (2010) conducted research on performance based multi-criteria decision making in bridge rehabilitation. The author presented a bridge repair decision framework, which considered a bridge as an element of a large transportation system. Bridge repair involves different stakeholders. The framework incorporates earthquake damage, seismic capacity, repair methods, construction workforce availability, business risks, and so on. Usually a transportation network includes more than one bridge; therefore, multiple bridge retrofit priority has to be determined. To reasonably distribute the funding for bridge retrofit, one crucial task is to rank the sequence of bridge retrofits. Therefore, bridge ranking is part of the performance based decision making. Traditionally, bridge ranking only includes factors like seismic risk, structural vulnerability, and bridge importance. Majid and Yousefi (2012) developed a ranking system to prioritize bridge repair sequence according to a number of criteria such as structural vulnerability, seismic hazard and importance classification. Wakchaure and Jha (2011) mentioned that bridge owners in India heavily rely on subjective judgment in allocating funding for bridge maintenance. To improve this situation, they used data envelopment analysis (DEA) method and scored 14 bridges in India. The analyzed variable included bridge health index, deck area and cost. They concluded that this approach is efficient and can provide better strategies than making subjective decisions.

Traditionally life cycle cost analysis is largely dependent on engineer’s experience. In order to improve the accuracy of the analysis and also keep it relative less sophisticated, Huang (2006) proposed a linear deterioration model supported by visual inspection.
inventory data. Liu and Frangopol (2005) presented a method that can optimize annual maintenance costs. The proposed model allows the bridge manager to choose maintenance methods by scientifically comparing different alternatives. When assessing the bridge health condition, bridge engineers usually rely on their experience and thus the results would be subjective. Rashidi and Gibson (2011) developed a rating approach which made the inspection more objective. In their study, they provided clear definition for different damage states. Analytic hierarchy process was used by the authors to evaluate the priority of parameters. Abu Dabous and Alkass (2008) also conducted a case study on bridge rehabilitation strategy using such a process. Analytic hierarchy process can be used to analyze problem by dividing it into small elements, thus make all criteria evident (Kabir et al., 2014). In the analysis, the authors considered cost, bridge safety, environment, and life span as variables.

Performance based decision making plays an important role in the post-earthquake bridge repair and retrofits (Gordin, 2010). Earthquake loss modeling involves the direct losses from structures and indirect losses from the interruptions of transportation. In many of current situations, the estimation of earthquake losses is still based on experience and is largely quantitative. More scientific engineering methods are to be developed to get accurate estimations. Many researchers have proposed earthquake loss models to solve this problem. Crowley et al. (2005) have analyzed the impact of uncertainty to earthquake loss models in various conditions. Cousins (2004) proposed a quick earthquake loss model that can cast light on losses efficiently. Lu et al. (2012) have developed loss fragilities based on PEER’s PBD framework. The variables in the fragility analysis
included earthquake force demand, damage and losses. In this fragility model, intensity measure (IM) is directly related to engineering demand (ED). The damage levels are categorized to two levels; structural level and transportation level. The structural level damage triggers the losses from bridge repair cost. This damage can be described as residual displacement, residual strain and maximum strains of bridge components. The transportation level damage indicates the bridge serviceability, which results in the indirect losses. In many places, it is expected that the indirect losses is greater than the direct losses. The damage to transportation level can be described as traffic carrying capacity and lane closures.

2.7 Soil structure interaction

Soil structure interaction (SSI) is one of the key components in performance based seismic design (Finn & Fujita, 2002; Priestley, 2000; Shamsabadi et al., 2007). The information of soil properties is usually provided by geotechnical engineers; therefore, close cooperation between structural engineer and geotechnical engineer is required. In PBD, structural engineers should not only consider the damage of structures. The damage of foundation and soil should also be considered. In the past events of Kobe and Christchurch earthquake, many structures were demolished because of the foundation level damages (Millen et al., 2014). Traditionally, soil structure interaction was regarded as one factor that benefits structural seismic response. Therefore, many design codes suggest neglecting soil structure interaction in order to generate a more conservative design. This is based on the three assumptions: (1) spectra acceleration decreases with increasing the period; (2) ductility factor is constant; (3) the damping from the soil is
correctly estimated (Mylonakis & Gazetas, 2000). However, from the findings by Mylonakis and Gazetas (2000) it was demonstrated that the increase in fundamental periods due to SSI does not necessarily lead to a mitigated structural response. The SSI from deformable soil may increase ductility demand significantly, which leads to a wrong direction of seismic design. In a later study by Jeremić et al. (2004), it was concluded that SSI can have both beneficial and detrimental effects depending on the characteristics of the earthquake. Therefore, SSI should be evaluated on a case by case basis. The methods of considering SSI in pile foundations and abutment-backfill system can be found in many literatures (Aviles & Pérez-Rocha, 2003; Boulanger et al., 1999; Shamsabadi et al., 2007; Spyarakos, 1992). Detailed procedures of incorporating SSI in PBD have also been presented by a number of researchers (Mekki et al., 2014; Roberts et al., 2010; Stewart et al., 2004).

One of the common practices of incorporating SSI in seismic design is to use p-y curves (Boulanger et al., 1999), where p strands for lateral soil pressure per unit length of the pile and y stands for the lateral deflection of the pile. The extensive usage of p-y method is due to its simplicity and ability to incorporate linear and non-linear analyses (Dash et al., 2008). The p-y method is briefly explained by using Figure 2.5 and Figure 2.6, where Figure 2.5 represents the global response of the structure and Figure 2.6 represents the local response of the soil spring. Response spectrum analysis is a linear analysis whereas p-y curves are non-linear. Therefore, several iterations have to be performed to find out the actual stiffness of the soil spring at the design level. At the beginning of the iteration, the stiffness and displacement of the spring can be assumed as K1 and d1. Then response
spectrum analysis is performed, which would generate another displacement $d_2$. If $d_2$ is different with $d_1$, then $d_2$ should be used in the local spring model to find out the corresponding stiffness $K_2$. The iteration should be continued until the stiffness and displacement from the global structure model converge with the local spring model.

Figure 2.5 Global response of structure

Figure 2.6 Local response of soil spring
2.8 Summary

PBD is a promising and complex design methodology. Great achievements have been made with efforts from numerous researchers and engineers in the past several decades. Many design codes have adopted PBD as a major design tool. A brief summary of the progress of PBD is presented in this study. It was also realized that some inconsistency may exist in the current codes and practices. The followings are the conclusions and recommendations for future research:

1. Displacement based design (DBD) is a promising design method under performance based design (PBD) framework. Numerous efforts from researchers and engineers have been made to expand the application of DBD.
2. Different design codes have various design criteria. Currently the CHBDC 2014 does not allow steel yielding for minimal damage at 1/475 year event for Major Route bridges. This requirement makes the CHBDC 2014 the most stringent code among all PBD codes considered in this study.
3. The current performance criteria are mostly for regular reinforced concrete and steel structures. PBD criteria need to be developed for different structures and systems, such as unbounded post-tensioned concrete structures and shape memory alloy reinforced concrete structures.
4. The next generation PBD is still going through evolution and getting matured over time. The PBD is now incorporating cost and time as other important factors in the design process. To properly estimate the repair cost and repair time, large amount regional data has to be made available. More research on optimizing
repair techniques and construction are needed. Governmental organizations, practitioners and researchers need to contribute to this area in a collaborative way.
Chapter 3: SEISMIC PERFORMANCE COMPARISON BETWEEN FORCE-BASED AND PERFORMANCE-BASED DESIGN OF A HIGHWAY BRIDGE AS PER CHBDC 2014

3.1 General

Bridges are critical civil infrastructure for transportation and economic development of a country. Seismic design is one of the most challenging parts during a bridge design, which is currently going through a transition phase in Canada. Conventionally, bridges are designed using force based design (FBD) method. This method calculates the seismic force demands by either single-mode or multi-mode spectral method for most of the bridge categories. For simple bridges, uniform-load method is often used, which is a simplified single-mode spectral method. The base shear force is reduced to the design base shear level using a force reduction factor R. Then the structure is designed according to this reduced force. However, the current FBD method has several shortcomings (Priestley et al., 2007a). The major limitation in the FBD method is that it cannot explicitly relate to the performance of the bridges as there are many uncertainties in achieving the expected level of performance. The second limitation is on its force-reduction factor R, which is utilized to scale down the seismic force. This R factor is based on ductility capacity and over-strength for a given structure type, which can vary significantly for similar type of structures with different geometry. Traditional FBD method ignores the fact that the displacement is more important than the strength for inelastic systems. However, displacement and deformation are the most direct reasons
that cause structural damages. Thirdly, design seismic force is applied to the structures with unchanged stiffness, which indicates that the elements of the structure will be subjected to yield at the same time. In reality, seismic force distribution is also affected by the yielding sequence of the piers. The stiffness of a structure is not constant as what is assumed in FBD, it changes with its deformation. In order to reduce the underlying uncertainties of FBD, researchers have developed the framework of PBD.

PBD has been developed to solve the above mentioned problems by researchers (FEMA, 1997; Poland et al., 1995; Priestley, 2000). Some of the design codes and guidelines such as CHBDC 2014 (CSA, 2014) have adopted PBD. PBD requires owners to select a target performance level and allows designers to directly control the performance of the designed bridge. The performance can be based on damage like drifts and strains. PBD has already been implemented in some large projects hoping to solve problems like uncertainty of performance (Marsh and Stringer, 2013). There are a number of bridges designed as per performance based approaches with at least two hazard levels (Jones et al., 2013; Khan & Jiang, 2015). At the lower level design, most of the bridges were designed for return period of less than 100 year, which included Tacoma Narrows Bridge, San Francisco to Oakland Bay Bridge, Evergreen Line and Gerald Desmond Bridge. In these designs, the target performance is essentially elastic where minor inelasticity may be expected. Willamette River Transit Bridge and Port Mann Bridge were designed for 1/475 year return period where the target performance was essentially elastic with minor inelastic response. Carquinez Straits Bridge was designed for 300 year return period, with
minor inelastic response. These design criteria were determined in order to achieve performances that meet the transportation and earthquake demand.

One reason for the popularity of PBD is that it can connect scientific design and seismic performance well so that decision maker can distribute resources more wisely. In terms of life safety in major earthquake, PBD is more prominent to determine seismic performance, while FBD frequently failed to provide expected performance. In several earthquakes like 1994 Northridge Earthquake, even though structures achieved the goals of protecting life safety, the costs of repair were unexpectedly high (Ghobarah, 2001). This is mainly because of displacement and deformation that determine seismic damages, rather than strength and capacity (Marsh & Stringer, 2013). Therefore, PBD can be a prominent method for future design of bridges.

Direct displacement-based design (DDBD) can be regarded as a PBD approach as shown to be prominent by many researchers (Adhikari et al., 2010; Alvarez Botero, 2004; Calvi et al., 2008). Some researchers have compared the designs between DDBD and FBD (Cardone et al., 2008; Correia et al., 2008; Reza et al., 2014). It was noted that the DDBD bridges may or may not perform better than the FBD bridge depending on different design cases. Reza et al. (2014) concluded that FBD bridges performed better than the DDBD bridges (while not considering confinement effect).

Since the design code allows both FBD and PBD for Major Route bridge design, it is necessary to compare the seismic performance of a bridge designed according to different
methods. This allows investigating the differences between the two methods when used in the design of actual bridges, which would also increase the confidence of bridge owners and designers to implement PBD in designs of highway bridges.

This research determines whether FBD is more conservative or un-conservative in comparison with PBD. Performance criteria in the CHBDC 2014 include strains of concrete and steel, the damage states of bearings and joints and other structural elements. The compared criteria used in this research are mainly material strain levels.

3.2 Performance objectives

PBD relates performance objectives with design process. Performance criteria can be based on any qualitative and quantitative response parameters such as strains and drifts. Performance objectives can be expressed in terms of specific damage states against prescribed probability demand levels (Ghobarah, 2001). For the specified response parameter criteria, the CHBDC (2014) uses material strains rather than drifts. This is due to the fact that drifts corresponding to specific damage states vary significantly with different systems. A good example may be the unbounded segmental post-tensioned concrete pier. A higher drift does not have to mean a more severe damage in this case. The unbounded segmental post-tensioned concrete piers may undergo 4% drift without any significant damage (Dawood et al., 2011) whereas traditional concrete structure (e.g. RC bridge pier) experiences extensive damage even below 4% drift. The damage states from CHBDC (2014) are briefly described in Table 3.1.
Table 3.1 Performance Criteria (CHBDC, 2014)

<table>
<thead>
<tr>
<th>Level</th>
<th>Service</th>
<th>Damage</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Immediate</td>
<td>Minimal Damage</td>
<td>Concrete compressive strains ($\varepsilon_c$) ≤ 0.004 and steel strains ($\varepsilon_{st}$) ≤ yield strain ($\varepsilon_y$).</td>
</tr>
<tr>
<td>2</td>
<td>Limited</td>
<td>Repairable Damage</td>
<td>Steel strains ($\varepsilon_{st}$) ≤ 0.015.</td>
</tr>
<tr>
<td>3</td>
<td>Service Disruption</td>
<td>Extensive Damage</td>
<td>Confined core concrete strain ($\varepsilon_{cc}$) ≤ concrete crushing strain ($\varepsilon_{cu}$). Steel strains ($\varepsilon_{st}$) ≤ 0.05.</td>
</tr>
<tr>
<td>4</td>
<td>Life Safety</td>
<td>Probable Replacement</td>
<td>Bridge spans shall remain in place but the bridge may be unusable and may have to be extensively repaired or replaced.</td>
</tr>
</tbody>
</table>

After determining performance levels at the beginning of the design, the performance criteria are assigned to different levels of earthquake events for different categories of bridges. The bridge category is usually defined based on the importance of the bridge. In the CHBDC (2014), there are three categories: Lifeline Bridges, Major-Route Bridges and Other bridges. The case study is a Major-Route Bridge. Major-Route Bridge is defined as the bridge that is a crucial part of regional transportation and is critical to post-disaster event and security. Based on the category of the bridge, performance levels are assigned to achieve the goals. In CHBDC (2014), the considered probabilities of exceedance are 2%, 5% and 10% in 50 years.

3.3 Case study description

The bridge is a multi-span concrete bridge with multi-column bents. The total span of the bridge is 100 meters and the width of the bridge is 40 meters. The bridge has 3 bents working as piers and 2 bents providing support as abutments. Each bent has 8 columns that are supported by single piles. The net height of each column is about 6 meters and
the length of each pile is 20 meters. Soil-structure interaction is considered in the bridge design and performance assessment. In the design phase, the bridge model was built in SAP2000 (CSI, 2010) and the soil-structure interaction was simulated by using a series of p-y springs. The finite element model of the bridge is shown in Figure 3.1. The plan view is shown in Figure 3.2. The bridge is located in Burnaby, Canada. Site-specific response spectra were used for the design. The spectra accelerations are shown in Figure 3.3.

Figure 3.1 Finite element model in SAP2000
In this case study, shallow soil is not strong enough to resist loads from the bridge; hence, pile foundations are used. The soil-structure interaction is an important factor that affects
the seismic performance of the bridge. In practice, the interaction between soil and structure is usually simulated by using p-y curves due to its simplicity (Dash et al., 2008). In p-y curves, p stands for lateral resistance force per unit pile length from soil, and y stands for lateral displacement of piles. Figure 3.4 shows a typical p-y curve where the soil loses its strength and stiffness with the increase of displacement.

![Figure 3.4 Typical p-y curve from field test](image)

3.4 Force-based design and performance-based design process

3.4.1 Force-based design process

In FBD, forces are calculated based on cracked stiffness, which can be estimated at the beginning of the design. Then, a force reduction factor is used to represent the ductility capacity. The reduced force is used for seismic design. Displacement of the structure is only checked at the end of the design. The flowcharts of FBD are shown in Figure 3.5.
The major steps and shortcomings in this process are discussed in details combining with the flowcharts.

In step 2, cracked stiffness is used to consider the reduction of stiffness. The stiffness is estimated based on axial load ratio and reinforcement ratio. The cracked stiffness can be found from the chart produced by Priestly (1996) or from moment-curvature analysis. FBD assumes that stiffness is independent of yielding strength. However, this might be problematic since strength and stiffness are coupled for many structures (Smith & Tso,
In step 4, the periods may be calculated from stiffness and mass of the structures by Equation 3.1.

\[ T = 2\pi \sqrt{\frac{m}{K}} \]  

where, \( m \) is the effective mass and \( K \) is the stiffness. Because the soil spring stiffness changes with the change of lateral load, the bridge has different fundamental periods at different earthquake events.

From step 4 to step 6, the bridge model was built in SAP2000 (CSI, 2010) for modal analysis and response spectrum analysis. Response spectrum analysis is a linear analysis, so that only linear soil spring can be used in the model. Since the soil loses strength and stiffness with the increase of lateral load, effective spring stiffness is used for the design. The effective stiffness of springs can be determined by conducting modal analysis and response spectrum analysis iteratively. At the beginning of the spring iterations, initial stiffness is defined and response spectrum analysis is conducted. The displacement of springs can be calculated from spectrum analysis and then another set of spring stiffness can be calculated. At different earthquake events, there should be different sets of soil springs that are iteratively determined from that event. Periods and elastic forces from acceleration spectrum are determined with the converged spring stiffness at the end.

In step 7, the ductility factor is defined by design codes and incorporated into the FBD. However, Priestly et al. (2007) mentioned that the force reduction factor differs
significantly in different codes and the definition of yield displacement is also different, thus may result in inconsistency and inaccuracy. For the design of bridges, it is assumed that all piers have equal displacement ductility demand. This assumption fits with regular bridges but may be unrealistic for irregular bridges. In step 8, base shear is distributed according to stiffness. This may also cause problems for irregular bridges since piers may not yield at the same time.

After designing the columns for moments with a code defined ductility factor, shear capacity can be determined by the lesser of elastic force or the actual force that causes the columns to form plastic hinges. The interaction between axial load, moment and shear should be considered. The over-strength of the columns is considered as the actual forces that cause plastic hinges. In the CHBDC 2014, plastic hinge moment is calculated as probable resistance, which is 1.30 times flexural expected resistance of concrete sections. The expected resistance is calculated with expected yield strength of reinforcement and expected concrete compressive strength.

3.4.2 Performance-based design process

In the CHBDC 2014, the PBD process can still largely be based on FBD. The major difference is that nonlinear time history or static pushover analysis is required to assess structural damages in PBD. Here, material strain is one of the most important criteria in determining seismic performance. For Major-Route Bridges, the damage should be limited within minimal, repairable and extensive damage levels in the event of 1/475,
1/975 and 1/2475 years. Figure 3.6 shows the flowcharts of PBD. The preliminary design results may be based on any design methods including FBD.

![Figure 3.6 Performance-Based Design flowcharts](image)

### 3.5 Design results

In CHBDC 2014, regular Major-Route bridges can be designed by using FBD method or PBD method, Lifeline bridges and irregular Major-Route bridges shall be designed by using PBD method. In this research, two FBDs were conducted as per CHBDC 2006 (CSA, 2006) and the CHBDC 2014 (CSA, 2014) respectively, which are denoted as D1 and D2. One PBD was conducted as per the CHBDC 2014 (CSA, 2014), which is denoted as D3. CHBDC 2014 requires that an Importance Factor of 1.5 is considered for Major-Route bridges in FBD. And spectral response acceleration values shall be 1/2475 years event. The three design results are shown in Table 3.2 and Figure 3.7.
### Table 3.2 Design cases

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Design method</th>
<th>Design Code</th>
<th>Column diameter (m)</th>
<th>Pier Longitudinal reinforcement ratio</th>
<th>Return period (years)</th>
<th>Longitudinal period (s)</th>
<th>Transverse period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>FBD</td>
<td>2006</td>
<td>0.914</td>
<td>1.9%</td>
<td>475</td>
<td>1.984</td>
<td>1.787</td>
</tr>
<tr>
<td>D2</td>
<td>FBD</td>
<td>2014</td>
<td>0.914</td>
<td>2.7%</td>
<td>2475</td>
<td>2.244</td>
<td>2.068</td>
</tr>
<tr>
<td>D3</td>
<td>PBD</td>
<td>2014</td>
<td>1.2</td>
<td>5.3%</td>
<td>475</td>
<td>1.598</td>
<td>1.362</td>
</tr>
</tbody>
</table>

![Figure 3.7 Column section](image)

Comparing the two FBDs, D2 has a higher reinforcement ratio due to the longer return period. It can be seen that the CHBDC 2014 results in higher reinforcement ratio comparing with CHBDC 2006. This conclusion may also apply to other similar design cases. However, the designed longitudinal reinforcement of D3 is extremely high, although the diameter of the column was increased to 1.2m to reduce displacement demands. This is mainly due to the requirement from the CHBDC 2014 that steel strains shall not exceed yield at 1/474-year event. Although D2 and D3 have different column sections and rebar ratio, they can be compared reasonably in terms of seismic capacity. Additionally, the construction cost of D2 would also be lower than D3.
3.6 Pushover analysis

To assess the performance of the bridge designed as per CHBDC 2006, pushover analysis was conducted on the transverse direction of each bent. Bents were pushed to the displacement demands calculated from response-spectrum analysis. The pushover analysis was performed using SeismoStruct (2014). Performance criteria such as strains can be directly shown in SeismoStruct (2014). For the nonlinear analysis, the concrete confinement model from Mander et al. (1988) and Menegotto-Pinto steel model (Menegotto and Pinto, 1973) were used. In Mander’s confined concrete model, the confinement is considered using one confinement factor. This is determined by the amount of lateral reinforcement and concrete properties.

D1 is designed as per CHBDC 2006 and its reinforcement ratio is 1.9%. The criteria from the CHBDC 2014 were used to assess its seismic performance. Transverse pushover analysis was carried out for each bent incorporating nonlinear p-y springs. Figure 3.8 to Figure 3.12 show the pushover curves with displacement demands and strain limits. The displacement demands from different events are shown with dashed vertical lines. The displacement demands were calculated from spectral analysis. Strain criteria are marked on the curves.
Figure 3.8 Abutment 0 pushover curve in D1

Figure 3.9 Bent 1 pushover curve in D1
Figure 3.10 Bent 2 pushover curve in D1

Figure 3.11 Bent 3 pushover curve in D1
As shown from Figure 3.8 to 3.12, all the bents reach yielding before 1/475-year event. Generally, the first yielding happens when bents reach about half of the displacement demands at 1/475-year event, which means that none of the bents meet the criteria from the CHBDC 2014 at 1/475-year event. For 1/975-year event, CHBDC 2014 requires that steel strains shall not exceed 0.015. Additionally, the concrete strain of 0.006 was also checked as a criterion for repairable damage. It was observed that Abutment 4 barely meets this requirement thus the bridge may reach extensive damage states at 1/975 years event. Abutment 4 shows damage much earlier than the other bents. This is because the soil conditions of abutments and piers are different. Pier 3 and Abutment 4 are supported by the weakest soil. The poor soil conditions at Abutment 4 lead to higher displacement demands and extensive damages. It was also found that all the bents can meet the criteria.
at 1/2475 years event since no significant strength degradation occurs and the steel strain of 0.05 was not reached. The performed pushover analysis shows that the bridge designed as per CHBDC 2006 is able to protect life safety for considered earthquake events. However, the repair cost may be very high due to the damages. This leads to the need of PBD.

To conduct performance-based design, nonlinear pushover or time-history analysis is required at design phase. In the PBD of this study, it was realized that Abutment 4 experiences the highest displacement demand and shows the most damage, so that pushover analysis was carried out only on Abutment 4. The pushover curve of Abutment 4 is shown in Figure 3.13. The CHBDC 2014 requires that steel strains shall not exceed yield at 1/475-year event. This requirement resulted in a very high longitudinal reinforcement ratio in piers, which was 5.3%. When the bent was pushed to the displacement demand, the maximum steel strain was 0.0024, which met the requirement of the CHBDC 2014. However, due to the high reinforcement ratio, the structure has huge amount of redundancy after the first yielding. At 1/975-year event, the concrete strain is even still smaller than 0.004, which is in minimal damage level. The steel strain only increases to 0.01 after 1/2475-year event, and concrete strain is smaller than 0.006 at 1/2475-year event. Even the steel strain of 0.015 was not reached through the analysis. It was also confirmed that if the requirements of 1/475 years are satisfied, the bridge may not even experience repairable damage level at 1/2475 years event. Comparing D3 with D1, D3 exhibits much more conservative design but may be challenging in terms of construction due to the high reinforcement ratio.
3.7 Performance assessment based on time-history analysis

To conduct a rigorous assessment on the seismic performance of D1, D2 and D3, time-history analyses are carried out. The performance criteria from the CHBDC 2014 were used for the evaluation. Before evaluating the three designs, it is important to explain in detail the reason for PBD. As shown in Figure 3.14, the most important criteria for PBD are structural safety and cost. It is obvious that structural safety have priority over cost for owner, consultant, contractor and other decision makers. However, a safe structure does not mean to have an extremely conservative design with high costs. Over-conservative designs may not only result in high costs but also being impractical to construct. It also brings more construction wastes and more environmental damages. Therefore, an optimized bridge design should not only focus on structural safety but also consider
reduction in construction and repair cost. Figure 3.14 shows a multi-criteria decision making on PBD. To protect structural safety and reduce repair cost, it is preferred that the structural damage is limited within an acceptable state. However, considering the feasibility of building such a structure, it is expected that the structure reaches corresponding damage state.

Figure 3.14 Multi-criteria decision making for performance-based design

In the time-history analysis, seven earthquake records were selected from The Canadian Association for Earthquake Engineering (Naumoski et al., 1988). The records were scaled based on site-specific response spectra using the program SeismoMatch (2014). This program uses the wavelets algorithm proposed by Abrahamson (1992) and Hancock et al. (2006). The input is the target spectrum at different design levels and the earthquake records. Acceleration loads were applied in both longitudinal and transverse directions. Table 3.3 lists the records selected for time-history analysis. It was determined that seven earthquake records are adequate for this case study because it was observed that the
material strain from the seven runs were very close to each other. However, if the results were dispersed, then more earthquake records would be needed.

Table 3.3 Earthquake records (Naumoski et al., 1988)

<table>
<thead>
<tr>
<th>Record Number</th>
<th>Earthquake Location</th>
<th>Date</th>
<th>Magnitude</th>
<th>Site Description</th>
<th>Max. Acc. A(g)</th>
<th>Max. Vel. V(m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley, CA</td>
<td>1940-5-18</td>
<td>6.6</td>
<td>El Centro</td>
<td>0.348</td>
<td>0.334</td>
</tr>
<tr>
<td>2</td>
<td>Kern County, CA</td>
<td>1952-7-21</td>
<td>7.6</td>
<td>Taft Lincoln School Tunnel</td>
<td>0.179</td>
<td>0.177</td>
</tr>
<tr>
<td>3</td>
<td>San Fernando, CA</td>
<td>1971-2-9</td>
<td>6.4</td>
<td>Hollywood Storage P.E. Lot, L.A.</td>
<td>0.211</td>
<td>0.211</td>
</tr>
<tr>
<td>4</td>
<td>San Fernando, CA</td>
<td>1971-2-9</td>
<td>6.4</td>
<td>Griffith Park Observatory, L.A</td>
<td>0.18</td>
<td>0.205</td>
</tr>
<tr>
<td>5</td>
<td>San Fernando, CA</td>
<td>1971-2-9</td>
<td>6.4</td>
<td>234 Figueroa St., L.A</td>
<td>0.199</td>
<td>0.167</td>
</tr>
<tr>
<td>6</td>
<td>Near East Coast of Honshu, Japan</td>
<td>1971-8-2</td>
<td>7</td>
<td>Kushiro Central Wharf</td>
<td>0.078</td>
<td>0.068</td>
</tr>
<tr>
<td>7</td>
<td>Monte Negro, Yugo</td>
<td>1979-4-15</td>
<td>7</td>
<td>Albatros Hotel, Ulcinj</td>
<td>0.171</td>
<td>0.194</td>
</tr>
</tbody>
</table>

Maximum strains from time-history analyses are presented in Tables 3.4 to 3.6 for three designs (D1, D2 and D3). It should be noted that only the results from the first 3 records are shown because of the limited space. Table 3.7 shows the damage states of the three designs determined from average strains of time-history analysis. From the time-history analysis, it was concluded that D1 fails to meet the criteria at 1/475-year event. This conclusion is similar with the findings from pushover analysis. D2 also fails to meet the criteria at 1/475-year event. D3 meets the criteria at all earthquake events and only reaches repairable damage states at 1/2475-year event. However, the issue with D3 is that the reinforcement ratio is too high. Although the maximum reinforcement ratio from the
CHBDC 2014 is 6%, which is higher than 5.3%, it makes concrete placement and proper vibration difficult. Another issue is that due to extreme conservative design, there is huge amount of redundancy in terms of residual capacity after the occurrence of first yielding. In the whole bridge structure, after one hinge is formed, there are still a large amount of plastic hinges to be formed. This results in a waste of resources and an increase in the cost of the structure. D1 tends to induce too much damage although life safety is protected. This will result in a very high repair cost. D3 tends to be too conservative with a huge amount of residual capacity. Considering the reinforcement ratio, proper construction may be very difficult. D2 is a design between D1 and D3, which may be the optimum choice in this study.

Table 3.4 Maximum strains of D1 from time-history analysis

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Material Damage</th>
<th>Earthquake record number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Concrete</td>
<td>1</td>
</tr>
<tr>
<td>475</td>
<td>0.003</td>
<td>0.003</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
</tr>
<tr>
<td>975</td>
<td>Concrete</td>
<td>0.004</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
</tr>
<tr>
<td>2475</td>
<td>Concrete</td>
<td>0.015</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Extensive</td>
</tr>
</tbody>
</table>

Note: $\varepsilon_y = 0.002; \varepsilon_{cu} = 0.019$
### Table 3.5 Maximum strains of D2 from time-history analysis

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Material</th>
<th>Damage</th>
<th>Earthquake record number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>475</td>
<td>Concrete</td>
<td>0.003</td>
<td>0.003</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.004</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
<td>Repairable</td>
</tr>
<tr>
<td>975</td>
<td>Concrete</td>
<td>0.004</td>
<td>0.004</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.006</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
<td>Repairable</td>
</tr>
<tr>
<td>2475</td>
<td>Concrete</td>
<td>0.007</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.013</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
<td>Minimal</td>
</tr>
</tbody>
</table>

Note: $\varepsilon_y = 0.002; \varepsilon_{cu} = 0.019$

### Table 3.6 Maximum strains of D3 from time-history analysis

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Material</th>
<th>Damage</th>
<th>Earthquake record number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>475</td>
<td>Concrete</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.0015</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Minimal</td>
<td>Minimal</td>
</tr>
<tr>
<td>975</td>
<td>Concrete</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.002</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Minimal</td>
<td>Minimal</td>
</tr>
<tr>
<td>2475</td>
<td>Concrete</td>
<td>0.003</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>0.004</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>Damage</td>
<td>Repairable</td>
<td>Minimal</td>
</tr>
</tbody>
</table>

Note: $\varepsilon_y = 0.002; \varepsilon_{cu} = 0.019$

### Table 3.7 Damage states of D1, D2 and D3

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>D1</th>
<th>D2</th>
<th>D3</th>
</tr>
</thead>
<tbody>
<tr>
<td>475</td>
<td>Repairable</td>
<td>Repairable</td>
<td>Extensive</td>
</tr>
<tr>
<td>975</td>
<td>Repairable</td>
<td>Repairable</td>
<td>Repairable</td>
</tr>
<tr>
<td>2475</td>
<td>Minimal</td>
<td>Minimal</td>
<td>Repairable</td>
</tr>
</tbody>
</table>
For the design of regular Major-Route Bridges, the CHBDC 2014 allows both PBD and FBD. However, it was found that the two methods are not consistent. Based on the findings from the time-history analyses of D2, a table of performance criteria is presented based on the maximum strains in earthquake events (Table 3.8). It shows what performance the prescribed FBD can achieve in CHBDC 2014.

Table 3.8 Performance of a bridge designed as per FBD in CHBDC 2014 (D2)

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Criteria</th>
</tr>
</thead>
</table>
| 475                   | Concrete compressive strains \( \varepsilon_c \) \( \leq 0.003 \)  
                       | Steel strains \( \varepsilon_o \) \( \leq 0.005 \) |
| 975                   | Concrete compressive strains \( \varepsilon_c \) \( \leq 0.005 \)  
                       | Steel strains \( \varepsilon_o \) \( \leq 0.008 \) |
| 2475                  | Concrete compressive strains \( \varepsilon_c \) \( \leq 0.008 \)  
                       | Steel strains \( \varepsilon_o \) \( \leq 0.014 \) |

### 3.8 Summary

Examples of typical highway bridge designs are presented in this section. The bridge was designed by using FBD as per the CHBDC 2006 (denoted as D1) and the CHBDC 2014 (denoted as D2), and also designed by PBD as per the CHBDC 2014 (denoted as D3). Site specific spectral accelerations and soil conditions were used in the design. The soil structure interactions were considered by using a series of p-y curves. D2 had a higher reinforcement ratio than D1. This is reasonable because the CHBDC 2014 is meant to improve structural safety. D3 had a much higher reinforcement ratio due to the strict requirements at 1/475-year event design. The 1/475-year event dominated the PBD.
After designing the bridge with three different approaches, pushover analysis and time-history analysis were conducted to evaluate its seismic performance. The results from pushover analyses and time-history analyses were similar in terms of damage states. It was found that D1 and D2 fails to meet the criteria at 1/475-year event. However, although D1 and D2 both met the criteria at 1/975 and 1/2475-year event, D2 showed much less damages than D1. Just comparing bridge performances based on damage states might be confusing and inaccurate. Because the steel strain value can range from 0.002 to 0.015 for repariable damage, this may represent very different repair costs. D3 met the criteria at all earthquake events. However, D3 requires a very high reinforcement ratio which makes concrete placement and proper vibration difficult. At the end, this study presented the performance that a bridge designed as per FBD in CHBDC 2014 can achieve. The following conclusions are made from this study.

1. Bridges designed as per CHBDC 2006 and CHBDC 2014 FBD are able to demonstrate good seismic performance in terms of protecting life safety.
2. Comparing with CHBDC 2006, CHBDC 2014 sets higher requirements on seismic design thus limits earthquake damages.
3. The PBD in CHBDC 2014 is conservative in comparison to FBD. This will certainly limit earthquake damage and repair cost, but may also increase initial cost and construction difficulties.
4. PBD only shows its prominent characteristics when the criteria are properly chosen. One cannot conclude whether PBD is more or less conservative than FBD unless PBD design performance is determined. Designing a bridge that remains
essentially elastic in 1/475-year event is able to limit earthquake damages. However, avoiding yielding of rebar will be challenging.

5. When performing PBD at different earthquake events, it is critical that the soil stiffness degradation be considered. This affects displacement demands significantly.
Chapter 4: DISPLACEMENT-BASED ANALYSIS ON CHBDC 2014

4.1 General

Performance based design (PBD) is of interest to researchers and engineers because it can make structural performance more predictable under predefined situations (Chen & Duan, 2014; Mackie & Stojadinović, 2007; Marsh, 2013). PBD is believed to be the next-generation design and assessment framework (Conte & Zhang, 2007). Although it has been proposed by researcher for a number of years, it was only recently adopted by design codes in recent years. For example, CHBDC (CSA, 2014) has incorporated PBD in 2014. It is characterized by criteria that can be directly measured and observed, such as displacements and strains. Whereas forces and stresses cannot be directly measured or observed. In the Newton's First Law of Motion, it has been stated that “every object in a state of uniform motion tends to remain in that state of motion unless an external force is applied to it” (Newton, 1999). It should be noted that force is the reason that causes rigid body motion but not damages. It is believed that deformation and strain are the direct reasons of structural damage. Displacement based design (DBD) is expected to be evolved to a better design method because it is realized that the traditional force based design (FBD) has a number of deficiencies (Priestley, 2000; Smith & Tso, 2002). For instance, in a traditional FBD, usually only the initial stiffness of the structure is used for the seismic design, which may underestimate displacement demands. Additionally, FBD assumes that all bridge piers have the same ductility. However, this assumption can be very inappropriate in terms of irregular bridges. In the case of multiple bent bridges with varied column lengths, the longer column should have a better ductility than a shorter
column. Because longer columns are flexural dominant and shorter columns are shear
dominant elements. FBD only assumes one single ductility factor based on structural type
of columns. Many of these shortcomings have been addressed by the DBD. DBD is the
most promising design method under the PBD framework because DBD method uses
target displacement to initial a design. The target displacement is a good measurement to
identify structural performance. Some other concerns such as ductility and stiffness have
also been better addressed in DBD.

4.2 Fundamentals of DBD
Displacement-based method was developed by a number of researchers (Calvi &
Kingsley, 1995; Kowalsky, 2002; Moehle, 1992; Pqulotto et al., 2007; Priestley, 1993;
Pristley & Calvi, 2007) to overcome the shortcomings of FBD. Displacement based
design was incorporated into AASHTO Guide Specifications for LRFD Seismic Bridge
Design (AASHTO, 2011; Chen & Duan, 2014). Similar to FBD, DBD is a general design
approach that can be adopted to all structural designs. The criterion for DBD is $\Delta D/\Delta C <
1.0$, where $\Delta D$ is displacement demand and $\Delta C$ is displacement capacity. Displacement
based design simplifies structures as a single degree of freedom system with a peak
displacement response. The philosophy in DBD is to design a structure which achieves a
certain performance limit state under a given situation.

The basic of displacement is based design is that the displacement capacity is more
important than the strength capacity (Pristley & Calvi, 2007). This is because the force
itself does not directly cause structural damage. It is the deformation that damages
structures. Although displacement based design methods were not used in the past, the ductility of structure was considered in the most design codes. The ductility of structure is the capacity of deformation after yielding without global failure (CSA, 2014). As long as bridge design avoids brittle failure such as shear failure and other construction deficiencies, it would have some ductility (Novakov et al., 2009). The construction deficiencies may include inappropriate rebar splice location at plastic hinge regions. Ductility of structures has three main advantages. Firstly, ductility of a structure increases its natural period and as a result, reduces the acceleration. Secondly, it can help structures absorb the seismic energy. Thirdly, it leads to yielding of members and accordingly limits the force in adjacent members.

In the displacement based design, the secant stiffness at the maximum displacement is used rather than the initial stiffness. A viscous damping factor is used to represent the elastic damping and the hysteretic energy absorption during inelastic responses (Priestley et al., 2007a). The first step of DBD is to determine target strains of materials at different limit states and drifts limits in order to estimate the design displacement. The relation between material strain and displacement can be identified using equations from Pristley and Calvi (2007). After determining the design displacement at the maximum response and the viscous damping, the effective period at the maximum displacement level can be calculated from displacement spectra. In CHBDC 2014 (CSA, 2014), displacement spectra can be calculated from acceleration spectra using Equation 4.1. In many codes such as Eurocode 8 (EC8) (de Normalisation, 1998), the displacement is constant beyond
a certain period (Bommer et al., 2000). The structural stiffness and the base shear can be calculated after determining the period.

\[ S_d(T) = 250 S(T)T^2 \]  \hspace{1cm} (4.1)

where, \( S(T) \) is spectra acceleration and \( T \) is period.

For multi-degree of freedom structures, the design displacement shape is based on the mode shapes at the design level. At different seismic design levels, the soil stiffness may change due to different foundation displacements. The mode shape will be determined by substructure and superstructure stiffness. Columns with less stiffness and abutments with less constrain will cause larger displacement. The design displacement at a pre-defined earthquake event is calculated from effective masses and displacements. Effective mass is calculated from each significant mass location and their displacement. Both superstructure and tall columns can be discretized to lumped masses. Superstructure includes all structural components above bearings such as girders and decks. At the same time, dead loads also need to be included as masses (e.g. the mass of asphalt for highway bridges and rail track for railway bridges). For irregular bridges with significant skew angles, the effect of torsional rotation should be considered under seismic loads. The following sections present a brief DBD procedure from Pristley and Calvi (2007).

### 4.3 Yield and design Displacement

Yield displacement is an important indicator of structural damage. It is an important factor in determining the ductility and the hysteretic damping. Priestley et al. (2007a) presented a number of equations for calculating the yield displacement under different
boundary conditions at column ends. For bearing supported superstructures, there would be less constraint compared with monolithic connected superstructure, where the girders and cap beams are connected in both longitudinal and transverse direction. This is usually constructed by encasing the girder into the cap beam, so that the columns are in double curvature in both horizontal directions. Thus, the yield displacement of piers equipped with bearings would be larger comparing to piers that are fixed at the top. Conditions of different foundations also influence the yield displacement in the same way, where less restraint will result in larger yield displacement.

For reinforced concrete structures, calculation of the yield displacement was investigated by Pristley and Calvi (2007). Pristley and Calvi (2007) concluded that yield curvature is a function of yield strain and section depth (Equation 4.2). By knowing the yield curvature ($\varphi_y$), the length of strain penetration ($L_{sp}$) and the column height ($H$), the yield displacement ($\Delta_y$) can be approximated using Equation 4.3. $\varepsilon_y$ is the yield strain and $D$ is the column diameter. It should be noted that in order to avoid steel yielding for Major Bridges and Lifeline bridges, according to CHBDC 2014, the yield displacement shall not be less than the displacement demand at 1/475-year event.

$$\varphi_y = \frac{2.25 \varepsilon_y}{D}$$

$$\Delta_y = \frac{\varphi_y(H + L_{sp})^2}{3}$$
The design displacement is usually determined by two considerations, which are strain limits and drift limits (Pristley & Calvi, 2007). In CHBDC 2014 (CSA, 2014), it is determined by strain limits. When strain limits govern design displacement, the design displacement is given by Equation 4.4, where $\Delta_y$ and $\Delta_p$ are yield displacement and plastic displacement, respectively. Otherwise if strain limits are not of concern, design displacement is determined by drift limit according to (Equation 4.5). The calculation of $\phi_{ls}$ depends on the failure type whether it is a compression failure (Eq. 4.6) or tension failure (Eq. 4.7). $\varepsilon_c$ and $\varepsilon_s$ are compression and tension strains, respectively. $c$ is the distance from neutral axis to the compression edge and $d$ is the distance from the neutral axis to the tension reinforcement. $\Delta_d$ is the design displacement. $H$ is the height of column and $\theta$ is the drift limit.

\[
\Delta_d = \Delta_y + \Delta_p \tag{4.4}
\]

\[
\Delta_d = \theta H \tag{4.5}
\]

\[
\phi_{ls} = \frac{\varepsilon_c}{c} \text{ for compression failure} \tag{4.6}
\]

\[
\phi_{ls} = \frac{\varepsilon_s}{d-c} \text{ for tension failure} \tag{4.7}
\]

### 4.4 Damping and ductility

Displacement based design uses secant stiffness and elastic displacement response spectrum. As a result, modification is required to consider the energy dissipation to accommodate inelastic energy dissipation. Although using inelastic displacement spectra might be easier to understand comparing with elastic displacement spectrum, it is a time-consuming process since spectra should be modified for each hysteretic rule (Priestly,
Therefore, Priestly (2007) suggested that the equivalent viscous damping (ζ), which combines the elastic damping (ζ_e) and the hysteretic damping (ζ_h) (Equation 4.8), is preferred. The hysteretic damping is calculated according to Equation 4.9. A is the area of force displacement response, F_m and Δ_m are the maximum force and displacement, respectively. The hysteretic damping is calculated from the area within one cycle of force-displacement response, maximum force and displacement.

\[ \zeta = \zeta_e + \zeta_h \]  
\[ \zeta_h = \frac{A}{2\pi F_m \Delta_m} \]  

For concrete and steel structures, the elastic damping ratio is 5% and 2%, respectively (SeismoStruct, 2003). The elastic damping describes the damping effect in the elastic behavior that is not captured by the hysteretic damping. Elastic damping is mainly caused by the interaction between structural and non-structural members and foundations. Elastic damping is defined as a fraction of critical damping, mass, and stiffness of the structure. Hysteretic damping is based on the energy absorption ability of hysteretic behavior. It should be noted that in CHBDC 2014, at the lower design level (1/475-year event) for Major Route Bridges and Lifeline Bridges, there is no hysteretic damping since the structure is supposed to be elastic.

To integrate the elastic and inelastic damping of pier and bearing, Equation 4.10 is suggested by Priestley et al. (2007). Δ_b is the displacement of bearing, Δ_ys is the yield displacement of pier, Δ_p is the plastic displacement of pier. The overall system damping
is weighted average of each pier and abutment. $\zeta_p$ is the overall damping of piers and $\zeta_B$ is the overall damping of bearings.

$$\zeta_e = \frac{\zeta_p (\Delta_y + \Delta_p) + \zeta_B \Delta_B}{\Delta_y + \Delta_p + \Delta_B} \tag{4.10}$$

### 4.5 Design base shear

The base shear is calculated from effective stiffness and design displacement (Equation 4.11 and Equation 4.12). Structural periods can be calculated by Equation 4.13. $T$ and $T_c$ are effective period and corner period, $\Delta d$ and $\Delta c$ are design displacement and displacement at the corner period. $\alpha$ is 0.5 and 0.25 for normal and velocity pulse conditions respectively.

$$V = K_e \Delta d \tag{4.11}$$

$$K_e = \frac{4\pi^2 m \Delta_c^2}{T^2 \frac{\Delta d}{\Delta c} \left( \frac{0.07}{0.02 + \zeta} \right)^2 \alpha} \tag{4.12}$$

$$T = T_c \frac{\Delta d}{\Delta c} \left( \frac{0.02 + \zeta}{0.07} \right)^\alpha \tag{4.13}$$

### 4.6 P-Δ effects

P-Δ effects are the additional overturning effects caused by weights of the structure (Davidson et al., 1992). In earthquake events, the lateral displacement caused by inertia force will cause second order effects which increase load demands. In the displacement based design proposed by Priestly et al. (2007), P-Δ effects are checked at the end of the
design. The effects are quantified by a stability index, which is the ratio of P-Δ moment and design base moment capacity. However, iterations may be needed to achieve a satisfactory stability index. Based on displacement based design method from Priestley et al. (2007a), Suarez and Kowalsky (2011) proposed a method that can consider the P-Δ effects at the beginning of the design and provide a limit to the design displacement. In their proposed method, they derived an equation for stability based design displacement to limit the stability index. This stability based design displacement would limit the P-Δ effect to acceptable limits. Therefore, no iteration is needed for the P-Δ effect calculation. However, they mentioned that this method is accurate in the case of regular continuous bridges and may not be applicable to other types of bridge.

4.7 Transverse displacement profiles

Dwairi and Kowalsky (2006) have investigated the inelastic displacement profiles of multi-span bridges, including rigid body translation, rigid body translation with rotation and flexible pattern. They concluded that the displacement profiles depend on the relative stiffness between superstructure and substructure, bridge regularity, and abutment type. Results showed that all four- and five-span symmetric bridges can be reasonably predicted using displacement based design. Some inaccuracies exist for long and irregular bridges. This is due to the fact that the effective mode shape fails to estimate the design displacement profile of irregular bridges.

Priestly et al. (2007) have discussed several possible displacement profiles for bridges. It was pointed out that displacement profiles can only be estimated at the beginning of the
design, and iteration is required to get accurate displacements. Then, the critical pier or abutment with the lowest capacity to demand ratio can be located. The displacement profile is calculated according to inelastic mode shape, design displacement, and modal value at the critical structural components. At the end, the design displacement of the substituted SDOF structure is calculated according to the mass and previous displacement profiles (Equation 4.14).

\[ \Delta_d = \frac{\sum m\Delta}{\sum m\Delta} \]  

(4.14)

where, \( m \) is the mass, \( \Delta_d \) is design displacement and \( \Delta \) is the displacement of structural components.

When the superstructure is simply supported and not constraint in rotation about the vertical direction, each pier can be designed individually as several SDOF designs. When the superstructure is continuous, the design can be classified to two different situations; rigid and flexible superstructures. The displacement profiles are straight lines for rigid superstructures. Depending on whether the bridge is symmetric or not, the displacement profile of each pier would be the same or different. When a movement joint exists, the displacement profile would comprise of two straight lines when superstructures are rigid. If superstructures are flexible, their shapes can be estimated by a sine or parabolic functions (Priestley et al., 2007a). When a movement joint exists, the displacement profiles would be two curves since the movement joint creates discontinuity.
4.8 Irregular bridges

Adhikari et al. (2010) mentioned that the displacement based design proposed by Priestly (2007) cannot properly capture displacement and base shear demands for long span bridges. The definition of long span bridges depends on structural type and engineering judgment. Generally, long span bridges are the structures that have small stiffness in the transverse direction and the dead load is usually more critical than live load. An extension of the displacement based design procedure was proposed by Adhikari et al. (2010) to consider issues related to straight long span bridges. The problems related to long bridges mainly include irregular flexural strengths of piers, incorporation of massive tall piers and effect of higher mode on the flexural strength of plastic hinges.

In the DBD method proposed by Priestley et al. et al. (2007), the shear force, V, is calculated as follows (Equation 4.15 and Equation 4.16):

\[
\text{Yield pier: } V \propto 1/H \quad \text{ (4.15)}
\]

\[
\text{Elastic pier: } V \propto u/H \quad \text{ (4.16)}
\]

where, H is the height of the pier, u is the displacement ductility demand. It is assumed that all piers have equal flexural strength. However, this may not be true, especially for long span bridges. Since the height of piers may vary significantly, the strength demands at pier bases may vary. The flexural strength may have a substantial amount of variation. The revised relations by Adhikari et al. (2010) are shown as in Equation 4.17 and Equation 4.18.

\[
\text{Yield pier: } V \propto \alpha/H \quad \text{ (4.17)}
\]
Elastic pier: $V \propto \alpha \frac{u}{H}$ \hspace{1cm} (4.18)

where, $\alpha$ is a ratio of flexural strength of a pier to the critical pier.

For irregular bridges, usually their center of mass does not coincide with the structure’s center of strength. This will result in rotational modes that increase seismic demands. The seismic response combines both translational and rotational modes of vibration (Restrepo, 2006). DBD has some limitations when it is implemented to irregular bridges since higher modes may play important roles in transverse response. Therefore, it would be difficult to estimate the displacement profiles (Kappos et al., 2012; Restrepo, 2006). Restrepo (2006) presented a number of examples of irregular bridges. It was concluded that some problems exist in terms of displacement pattern when DBD is implemented to very stiff bridges. The reason was that their first elastic and inelastic mode shapes are different. To solve this problem, he suggested that redistributions of strength and stiffness can improve the performance of bridges. Reza et al. (2014) did a comparison between DBD and FBD of a multi-span irregular bridge with piers having short periods. Their results suggested that in terms of displacement demand, residual displacement and energy dissipation, FBD was better than DBD in the case where confinement was not considered. They indicated that the DBD process works efficiently for the design of symmetrical bridges, while showing some shortcomings when it is applied to highly irregular bridges.
4.9 Displacement based analysis

To incorporate displacement based analysis in CHBDC 2014, an example is presented in this section. In this case study, a single degree of freedom bridge is designed for the purpose of demonstration. The bridge is classified as a Major Route bridge, assuming a footing-supported column monolithically connected to the superstructure. A two level design is performed as per CHBDC 2014. The design parameters are listed in Table 4.1.

Table 4.1 Design parameters

<table>
<thead>
<tr>
<th>Column diameter</th>
<th>Column height</th>
<th>Concrete strength</th>
<th>Rebar yield strength</th>
<th>Concrete cover thickness</th>
<th>Spiral spacing</th>
<th>Axial load ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>914 mm</td>
<td>6 m</td>
<td>35Mpa</td>
<td>400Mpa</td>
<td>70 mm</td>
<td>75 mm</td>
<td>10%</td>
</tr>
</tbody>
</table>

For Major Route bridges, 1/975 year-event criteria is optional unless required by the Authority having jurisdiction or Owner. The bridge shall be designed for 1/475 and 1/2475-year events. At 1/475-year event, the concrete strain shall not exceed 0.004 and the steel strain shall not exceed yielding (0.002). At 1/2475-year event, steel strain shall not exceed 0.05 and concrete shall not crush.

4.9.1 Determination of the material which governs the design at 1/475-year event

At the lower level design, CHBDC has requirements for both concrete and steel strains. Thus the first step may be to determine which material governs the design. At the beginning of the design, 2% reinforcement ratio is assumed. The reinforcement ratio is defined as the ratio of longitudinal rebar area to concrete gross section area.
To determine which material governs the design, a simple section analysis is first conducted. Section analysis is the process of calculating the strength and strain of reinforced concrete sections. From the section analysis, it was shown that when steel strain reaches 0.002, the concrete strain is lower than 0.004. Therefore, steel strain governs the design. Table 4.2 shows corresponding concrete and steel strain values for columns with 1% and 2% rebar ratio. The calculation was performed by using XTRACT (Chadwell & Imbsen, 2004). From Table 4.2, it can be seen that concrete strain can hardly govern the design. When concrete strain reaches 0.004, steel strains are around 0.01. When steel reaches yielding, the concrete strain is between 0.0012 and 0.0014.

<table>
<thead>
<tr>
<th>1% reinforcement ratio</th>
<th>2% reinforcement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Steel</td>
</tr>
<tr>
<td>0.00127</td>
<td>0.002</td>
</tr>
<tr>
<td>0.0026</td>
<td>0.005</td>
</tr>
<tr>
<td>0.0039</td>
<td>0.01</td>
</tr>
<tr>
<td>0.006</td>
<td>0.015</td>
</tr>
<tr>
<td>0.004</td>
<td>0.011</td>
</tr>
<tr>
<td>0.005</td>
<td>0.013</td>
</tr>
<tr>
<td>0.007</td>
<td>0.025</td>
</tr>
<tr>
<td>0.015</td>
<td>0.05</td>
</tr>
</tbody>
</table>

It would be of interest to perform a calculation in order to observe when the concrete strain governs the design. For the demonstration purpose and simplicity, a square section is assumed. The section is 1m*1m, concrete strength is 35Mpa, steel yielding strain is
0.002. The following calculation determines which material governs design at 1/475-year event. At the two limit states, the smaller curvature controls the design.

\[ \varphi_c = \frac{\varepsilon_c}{c} = \frac{0.004}{c} \]  
(4.19)

\[ \varphi_s = \frac{\varepsilon_s}{d-c} = \frac{0.002}{1-c} \]  
(4.20)

\( \varphi_c \): concrete curvature; \( \varphi_s \): rebar curvature; \( \varepsilon_c \): concrete strain; \( \varepsilon_s \): steel strain; \( d \): effective depth; \( c \): distance from the extreme compression fiber to the neutral axial.

By solving the equation \( \varphi_c = \varphi_s \), \( c \) equals to 0.66. This means that when \( c \) is greater than 66% of the total depth, the concrete strain governs the design. In this case, \( \alpha_1 \) equals 0.8 and \( \beta_1 \) equals 0.88, where \( \alpha_1 \) is the ratio of average stress in a rectangular compression block to the specified concrete strength and \( \beta_1 \) is a concrete compressive stress area factor. Therefore, axial load ratio equals \( \frac{A_g f'_c \beta_1 \alpha_1 c/d}{A_g f'c} (0.465) \). This means that only when axial load ratio reaches 0.465, the concrete starts to govern the design. However, it is not likely that the axial load ratio would be 0.465 at 1/475-year event since the column is only designed for 0.1 axial load ratio; therefore, steel strain governs the design.

### 4.9.2 Displacement based design based on steel strain

The confined concrete strength (\( f_{cc} \)) can be estimated through from the chart provided by Priestley et al. (2007), or using the equations from Mander et al. (1988). Using the equations from Mander et al. (1988), it was calculated that \( f_{cc} = 1.3f'_{ce} = 59.15 \text{MPa} \). By
assuming a concentrated plastic hinge and using section analysis, the plastic deformation can be estimated. Neutral axis depth can also be estimated from the chart provided by Priestley et al. (2007) or by performing a section analysis, both methods were used in this study. The axial load of the column is assumed as 2300kN, which is 10% of the axial load capacity. c/D is 0.25 based on the charts from Priestley et al. (2007), where c is the distance from the neutral axis to the compression edge, and D is the column diameter. It was calculated that $\varnothing_{s-2475} = 0.08$, $\varnothing_{y-475} = 0.0049$, and the plastic hinge length is 580mm, $\Delta_y = 0.035$ and $\Delta_d = 0.29$. Therefore, the displacement ductility capacity ($\Delta_d/\Delta_y$) based on the strain values is 7.

Displacements corresponding to different strains are shown in Table 4.3. The relation between displacement and drift is also presented in Figure 4.1. It can be seen that the displacement of pier increases linearly with the increase of steel strain. This is because strain and curvature are in a linear relation and the design displacement increases linearly with the increase of curvature assuming that the plastic hinge length is fixed. The calculated displacement is consistent with the results from Priestley et al. (2007). The displacement ductility of the column is 7 based on the two strain values of 0.002 and 0.05.
Table 4.3 Column strain and displacement

<table>
<thead>
<tr>
<th>Steel Strain</th>
<th>Curvature</th>
<th>Displacement (m)</th>
<th>Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.002</td>
<td>0.005</td>
<td>0.04</td>
<td>0.60%</td>
</tr>
<tr>
<td>0.005</td>
<td>0.01</td>
<td>0.05</td>
<td>0.89%</td>
</tr>
<tr>
<td>0.01</td>
<td>0.019</td>
<td>0.08</td>
<td>1.41%</td>
</tr>
<tr>
<td>0.015</td>
<td>0.028</td>
<td>0.12</td>
<td>1.94%</td>
</tr>
<tr>
<td>0.02</td>
<td>0.034</td>
<td>0.14</td>
<td>2.28%</td>
</tr>
<tr>
<td>0.05</td>
<td>0.08</td>
<td>0.30</td>
<td>4.95%</td>
</tr>
</tbody>
</table>

Figure 4.1 Displacement-strain relations

For the structures with period ranges from 0s to 5s, the ductility demand can be calculated by dividing the displacement demand at 1/2475-year event by the displacement demand at 1/475-year event. The ductility demand is around 2 to 3, which is much smaller than the ductility capacity of the structure in this case. Therefore, 1/475-year event governs the design. After determining the effective period, corresponding stiffness and base shear can be calculated based on the displacement spectra (Figure 4.2) using Equation 4.11 and Equation 4.12.
4.10 Relation between strain and drift

This section presents the relation between strain and drift of monolithic connected columns with varying section diameter, column height, and reinforcement ratio ($\rho_l$). It is assumed that the steel strain governs the design. The results for columns with 2% and 4% reinforcement ratio are shown in Figure 4.3. It can be seen that column drift increases linearly with the increase of steel strain. These charts can aid engineers to choose the diameter of columns at the beginning of the design. By determining drift and strain limits, the diameter can be estimated. Simple linear interpolation can be used when other design parameters are used.

It can be concluded that column diameter is the most significant factors that affects drift - steel strain relation. A larger diameter not only results in earlier yielding, but also higher plastic strain at the same drift level. At the same strain value, higher reinforcement ratio and taller columns result in larger displacement.
Figure 4.3 Drift-Steel strain relations for various columns
4.11 Implication and discussion

Table 4.4 shows the acceleration and displacement spectra for some Canadian cities including Toronto, Montréal, Calgary, Ottawa, Edmonton, Winnipeg, Mississauga and Vancouver. The displacement spectra are also plotted in Figure 4.4. It can be seen that at the high level design, the displacement demand is generally 1.9 to 2.6 times of that at 1/475-year event assuming the time period of the low level design is close to the high level design. Although the periods at low and high level designs are different, the two periods should be close. This means the ductility demand ranges from 1.9 to 2.6. However, according to displacement based design approach, from the strain of 0.002 to 0.05, the ductility of RC columns ranges from 5 to 7. Since the ductility demand is much lower than the ductility capacity, it can be concluded that the lower level event (1/475-year) governs the design. The lower level design poses very strict limit in high seismic regions. Strong columns may not only cause construction difficulties, but also pose trouble to superstructure design because it is preferable to have plastic hinges at columns rather than beams.
Table 4.4 Acceleration and displacement spectra at T=1s (class C soil)

<table>
<thead>
<tr>
<th>City</th>
<th>Longitude</th>
<th>Latitude</th>
<th>Acceleration spectra at different Return periods (g)</th>
<th>Displacement spectra at different Return period (mm)</th>
<th>Ductility Demand (T=1s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toronto</td>
<td>43.70</td>
<td>-79.40</td>
<td>0.066 24575 year 0.027 457 year</td>
<td>16.5 2475 year 6.75 457 year</td>
<td>2.44</td>
</tr>
<tr>
<td>Montréal</td>
<td>45.50</td>
<td>-73.56</td>
<td>0.137 2475 year 0.057 457 year</td>
<td>34.25 34.5 year 14.25 14.25 year</td>
<td>2.40</td>
</tr>
<tr>
<td>Calgary</td>
<td>51.04</td>
<td>-114.0</td>
<td>0.041 457 year 0.02 24575 year</td>
<td>10.25 10.25 year 5 5 year</td>
<td>2.05</td>
</tr>
<tr>
<td>Ottawa</td>
<td>45.42</td>
<td>-75.69</td>
<td>0.138 24575 year 0.056 457 year</td>
<td>34.5 34.5 year 14 14 year</td>
<td>2.46</td>
</tr>
<tr>
<td>Edmonton</td>
<td>53.53</td>
<td>-113.5</td>
<td>0.026 2475 year 0.01 457 year</td>
<td>6.5 6.5 year 2.5 2.5 year</td>
<td>2.60</td>
</tr>
<tr>
<td>Winnipeg</td>
<td>49.89</td>
<td>-97.13</td>
<td>0.026 2475 year 0.011 457 year</td>
<td>6.5 6.5 year 2.75 2.75 year</td>
<td>2.36</td>
</tr>
<tr>
<td>Mississauga</td>
<td>43.60</td>
<td>-79.65</td>
<td>0.064 2475 year 0.026 457 year</td>
<td>16 16 year 6.5 6.5 year</td>
<td>2.46</td>
</tr>
<tr>
<td>Vancouver</td>
<td>49.28</td>
<td>-123.12</td>
<td>0.333 24575 year 0.172 457 year</td>
<td>83.25 83.25 year 43 43 year</td>
<td>1.94</td>
</tr>
</tbody>
</table>
In CHBDC 2014, a Major Route bridge may be designed by using FBD for 1/2475-year event. In the FBD, a force reduction factor of five may be used to scale down the seismic force for multi-bent concrete bridges (CSA, 2014). This is equivalent to reaching the design displacement to a value five times of the yielding displacement. Meanwhile, as
long as a bridge meets the criteria at 1/475-year event, it automatically meets the criteria at 2475 years event requirements. Since the displacement demand at 1/2475 year event is only two to three times of that at 1/475 year event.

4.12 Summary

This chapter presented an example DBD for CHBDC 2014 by following the procedure provided by (Pristley & Calvi, 2007). The following findings are concluded:

1. For irregular bridges, current displacement based design (DBD) procedures have several shortcomings which need to be addressed in order to perform a better design.
2. The DBD procedure proposed by (Pristley and Calvi (2007)) is compatible with the current CHBDC 2014.
3. The PBD from CHBDC 2014 is highly conservative compared to the previous version of CHBDC. The current PBD is also more conservative than the current FBD. This is because, when a bridge is designed as per PBD the force reduction factor is two to three, while in the case of using FBD this factor is five. FBD allows scaling down seismic force using a factor of five whereas the same reduction is not permitted in PBD.
4. The displacement demand for many cities in Canada at 1/2475-year event is 2 to 3 times to that of the demand at 1/475-year event. This leads to the fact that 1/475-year event tends to govern the design of ductile structures.
Chapter 5: PERFORMANCE BASED DESIGN CONSIDERING REPAIR COST AND TIME

5.1 General

Performance based design (PBD) not only shows advantages over force based design (FBD) in structural designs, but also has economic benefits in the long term life cycle costs (Gordin, 2010). One of the most important incentives for developing the PBD is to reduce the repair cost and repair time. Therefore, considerations on repair cost and repair time should be incorporated as a part of the project design phase. PBD may be the most popular structural design approach under the Public–private partnership (PPP) framework. PPP is the partnership funded and invested by both the government and private sectors (Ke et al., 2010; Samii et al., 2002). Under PPP framework, owners and engineers have a better cooperation and an easier understandable design approach (Abdel, 2007). Many of the bridge projects were developed under the Public-private partnership. Under the Public-private partnership framework, different sectors such as bridge owners, consultants and constructors have more flexible negotiations and they also share the risks. There are a number of project deliver models under PPP framework, such as Design Build, Design Build Operate Maintain, Design Build Finance, and Design Build Finance Operate Maintain. Differences between these models are briefly explained here.

In the Design Build method, the owners only sign one contract that combines both engineering design and construction. The private section can be comprised by one or
several companies. A recent Design Build project in British Columbia, Canada, is the Port Mann Highway 1 Improvement Project. In this project, a Design Build contract was signed between the Province of British Columbia and the private sector Kiewit/Flatiron. The signed contract was $2.46 billion and any cost overruns was the responsibility of the contractor (Shamloo, 2014). Design Build Operate Maintain is a deliver model that incorporates operations and maintenance with design and build (FHWA, 2015). In this model, the public sector signs one contract and secures the financing, then the operation, and maintenance are achieved by the private sectors. Under the Design Build Finance framework, the private sector has the responsibility of design, construction and financing of the infrastructure such as bridges. This is often used when the owner has the desire to deter the payment. For the Design Build Finance Operate Maintain method, all the five responsibilities are taken by the private sector. It is also common that the contracts last for a long time period, rather than just the design and construction phase. Therefore, the life cycle cost of the projects are considered at the beginning. A number of researchers have investigated the earthquake impact on the life cycle analysis (Chang & Shinozuka, 1996; Fragiadakis et al., 2006; Liu et al., 2003; Takahashi et al., 2004). Life cycle analysis is usually performed to estimate the long term cost of a project including initial construction, inspection, and repair. The target is to minimize the total life-cycle cost while maintaining structural safety (Frangopol et al., 2001). The PBD will bring many benefits to the life cycle cost of bridge projects. Incorporating repair cost and repair time in the design phase is the main task of the next generation PBD. Under the next generation PBD, a program named Bridge PBEE was developed by the Pacific Earthquake Engineering Research Center (PEER) (Lu et al., 2011). In this chapter, as a
case study, the program Bridge PBEE is used for the performance based earthquake engineering analysis in order to compare bridges having different reinforcement ratios and column diameters.

The repair time in this study is expressed in terms of working days. It is simply the summation of repair time of each components such as columns, bearings and abutments. The total repair time does not equal to construction durations since the repair time here does not consider concurrent constructions. Additionally, the repair construction speed is assumed to be constant irrespect of resources and constraints (Lu et al., 2011). It will still be the responsibility of the project engineer to determine the actual construction durations. However, as long as total the repair time can be estimated, the construction durations can be easily estimated based on available resources. The construction durations are the ratios of total repair time to available construction speed.

When calculating repair cost, the double counting of repair quantities are avoided in this program. For example, the soil excavation for the back wall repair and the foundation repair is performed only for one time. It is relatively easier to avoid double counting repair cost than repair time, since it only dependens on the structures rather than available sources. The construction schedule is highly dependent on available resources and restraints from the natural and social environment.
5.2 Fragility analysis

The case study is a two-span reinforced concrete bridge. The models have multiple degrees of freedom incorporating bridge piers and abutments. Three designs are compared based on the next generation PBD framework. Differences between the three designs are reinforcement ratio and column diameter. The three different reinforcement ratios are 1.9%, 2.7%, and 5.3%; the corresponding designs are named Design I, Design II and Design III. Additionally, the column diameter of Design I and Design II is 0.914m and the column diameter of Design III is 1.22m. Fragility analysis is performed to compare the seismic performance of the three designs. It is common to compare the seismic performance using nonlinear pushover analysis and time-history analysis. However, these methods are limited to the comparison of structural damages. To perform a more detailed comparison, fragility analysis can be used. Furthermore, fragility analysis does not have to be limited in presenting damage states of the structure. Fragility curves can also be presented in terms of economic losses. In the past, many fragility curves show the first two integrations in Equation 5.1, which ends at DM (damage measurement). However, in this research, one more integration is considered, which is the DV (decision variable). DV is expressed in terms of repair time and cost. They are calculated from the damage state of each structural component.

\[
P(D > dv) = \int_{dm} \int_{ep} \int_{im} G_{DV} \cdot dG_{DM} \cdot dG_{EDP} \cdot dG_{IM} / \]

84
where, \( P \) is the total probability of a structure exceeding limit states, \( DM \) is damage measure, \( EDP \) is engineering demand parameter, \( IM \) is seismic intensity measure, \( DV \) is decision variables.

Fragility analysis determines the probability of structures achieving a certain damage state in a prescribed ground motion intensity measurement (Billah & Alam, 2014a). The probability of exceeding the limit states can be expressed as Equation 5.2:

\[
\text{Fragility} = P(\text{DS} \mid \text{IM} = y) \tag{5.2}
\]

where, \( DS \) is the damage states, \( IM \) is the intensity measurement and \( y \) is the given intensity measure of a considered ground motion. The fragility curves can be developed by either experimental data or by analytical simulations. When analytical approaches are used, two different methods are available, which are called incremental dynamic analysis and cloud method. The incremental dynamic analysis is a dynamic equivalent method to pushover analysis. In the incremental dynamic analysis, a number of ground motions are scaled to several selective intensity levels (Vamvatsikos & Cornell, 2002). Alternatively, the analysis can be performed using cloud method. The cloud method has been used in this study to capture a wide range of earthquake intensities. The cloud method is not only able to cover a wide range of peak ground accelerations but also epicenter distances. When using cloud method, the selected earthquake records represent a variety of intensity measures so that they do not need to be scaled (Mackie & Stojadinovic, 2005). After performing the time history analysis based on the cloud approach, regression analyses are
performed. It is assumed that a two-parameter lognormal probability distribution can be used by defining a median and a standard deviation. The power law function can be expressed as Equation 5.3.

$$\text{EDP} = a \cdot (\text{IM})^b \text{ or } \ln(\text{EDP}) = \ln(a) + b \ln(\text{IM})$$

(5.3)

where, EDP is an engineering demand parameter such as drift ratio or steel strain, IM is an intensity measurement such as peak ground acceleration or velocity, a and b are parameters determined from the regression analysis.

5.3 Earthquake records

In this study, a 100 earthquake motion ensemble by Mackie and Stojadinović (2005) is used. The 100 earthquake records are categorized into bins: the characteristics of the bins are shown in Table 5.1. In the earthquake bins, large magnitude is defined as a moment magnitude that are higher than 6.5 M. Short distance is defined as an epicenter distance smaller than 30 kM but greater than 15 kM. Near field is defined as smaller than 15 kM epicenter distance. Long distance earthquake is defined as epicenter distance beyond 30 but within 60 kM.
Table 5.1 Earthquake record bins

<table>
<thead>
<tr>
<th>Bin No.</th>
<th>Moment magnitude (Mw)</th>
<th>Closest distance (R), kM</th>
</tr>
</thead>
<tbody>
<tr>
<td>LMSR ground motion</td>
<td>6.5-7.2</td>
<td>15-30</td>
</tr>
<tr>
<td>LMLR ground motion</td>
<td>6.5-7.2</td>
<td>30-60</td>
</tr>
<tr>
<td>SMSR ground motion</td>
<td>5.8-6.5</td>
<td>15-30</td>
</tr>
<tr>
<td>SMLR ground motion</td>
<td>5.8-6.5</td>
<td>30-60</td>
</tr>
<tr>
<td>Near (field) ground motion</td>
<td>5.8-7.2</td>
<td>0-15</td>
</tr>
</tbody>
</table>

Note: LMSR= Large Magnitude–Short Distance Bin, LMLR= Large Magnitude–Long Distance Bin, SMSR= Small Magnitude–Short Distance Bin and SMLR= Small Magnitude–Long Distance Bin

The motions are selected randomly from each bin and 20 motions are selected for each bridge. The motions are comprised of two horizontal records and one vertical record. Each of the three bridges is analyzed by running 20 time history analyses. The amount of records is adequate since the selected records cover the range of interested peak ground accelerations (PGA) in British Columbia, Canada, where the PGA ranges from 0.116g to 0.455g at different earthquake levels (NRC, 2015). The histograms of PGA in a ground motion set are shown in Figure 5.1 to 5.3 for three directions. They show the distribution of PGA in a data set. In the longitudinal direction, the PGAs range from 0.03g to 0.57g. In the transverse direction, the PGAs range from 0.02g to 0.42g with one exception which is 0.8g. In the vertical direction, the PGAs range from 0.01g to 0.51g. For transverse and longitudinal records, most of PGAs are in the range between 0.03g and
0.4g. The maximum PGA in transverse direction is 0.84g. For the vertical records, most of PGAs are below 0.2g.

Figure 5.1 Longitudinal PGA Histogram

Figure 5.2 Transverse PGA Histogram
In all three directions, PGA cumulative distribution curves are plotted. From Figure 5.4 to Figure 5.6. In the longitudinal PGA cumulative distribution, it can be seen that 80% of the PGAs are below 0.4g. In the transverse GPA cumulative distribution graphs, 90% of the PGAs are within 0.4g. In the vertical GPA cumulative distribution graphs, about 80% of the PGAs are below 0.2g. It can be seen that the selected motions represent a variety of earthquake records. This is a prerequisite for the cloud method based fragility analysis.
Figure 5.4 Longitudinal PGA Cumulative Distribution

Figure 5.5 Transverse PGA Cumulative Distribution

Figure 5.6 Vertical PGA Cumulative Distribution
5.4 Performance comparison

The performance comparisons in this section include engineering parameters and decision variables. EDPs are various maximum displacement and residual displacement. The decision variables include repair cost and repair time. This is performed under the next generation PBD framework (Hamburger et al., 2004).

Under the framework of PBEE analysis, the direct structural responses are defined as engineering demand parameters for each performance group (PG), which are calculated from intensity measures (IM). The performance groups include: 1) maximum column drift ratio, 2) residual column drift ratio, 3) maximum relative deck-end/abutment displacement (left), 5) maximum relative deck-end/abutment displacement (right), maximum bridge-abutment bearing displacement (left), 6) maximum bridge-abutment bearing displacement (right), 7) approach residual vertical displacement (left), 8) approach residual vertical displacement (right), 9) abutment residual pile cap displacement (left), 10) abutment residual pile top displacement (right), and 11) column residual pile displacement at ground surface. For the two residual pile cap displacements at abutments and one residual pile cap displacement at column, the square root of the sum of the squares rule is used to consider the multiple modes of the structure. All these structural responses are the direct output from the time history analysis. Among these responses, performance groups 1 and 2 can be used to identify column damage states. Performance groups 3, 4, 5 and 6 can be used to define the damage states of abutments. Performance groups 7 and 8 can be used to define the damage states of approach. Performance groups 9, 10 and 11 are used to define damage states of the foundations.
The performance groups are only selected for quantities that affect repair cost and time significantly. One important function of categorizing performance group is to eliminate the double counting repair items. For example, back wall excavation is needed for both abutment repair and back wall repair. In such cases, the back wall excavation cost and time should only be count once.

After running the time history analysis based on the cloud method, the engineering demands and the intensity measure are the direct outputs. The output can be plotted in a log-log scale and a probabilistic seismic demand model (PSDM) model can be developed.

The performance group response versus GPA of the bridge Design I (1.9% rebar ratio) is shown in Figure 5.7. Figure 5.7 shows the relation between column drift ratio and PGA. In this figure, the average value, -1 sigma and +1 sigma are plotted. Sigma is the standard deviation in the normal distribution. The curve with -1 sigma and +1 sigma sign present the values that is one standard deviation smaller and greater than the expected values, respectively. Additionally, the results from different bins are also plotted, where LMSR is Large Magnitude–Short Distance Bin, LMLR is Large Magnitude–Long Distance Bin, SMSR is Small Magnitude–Short Distance Bin and SMLR is Small Magnitude–Long Distance Bin. By comparing the results from different bins, it can be concluded that LMSR and near field motions have more damaging effect on columns. This is because the epicenter distance is a major factor that affects damage caused by earthquake. The damage caused by SMSR and SMLR are relatively smaller. The damage extent of LMLR
is in the middle of all the data sets. This observation also confirms the importance of selecting earthquake motions that are representative of region characteristics.

In order to compare the seismic performance of the three bridges, their engineering demand parameters are plotted in Figures 8 to 13. It can be concluded that Design I and Design II have similar maximum drifts although Design I has slightly higher damages, Design III has much smaller residual drift. The rebar ratios of the three designs are 1.9%, 2.7%, and 5.3% respectively. However, the maximum drift is mainly a function of structural mass and stiffness. The difference in rebar ratio between Design I and Design II is 0.8%. This small difference does not make a significant change in structural mass or stiffness. The different rebar ratio is more critical in the capacity side rather than the displacement demand side. The reason that Design III has much small maximum drift is
mainly due to the increase in stiffness of columns. From Figure 5.9, it can also be seen that the Design II only shows slightly smaller residual displacement than Design I, whereas Design III show much smaller values. Residual displacement is caused by the concrete cover spalling, concrete core crush and the inelastic behavior of steel rebar. Residual displacement is not only a function of rebar ratio, but also a function of re-centering capacity. Comparing Design I and Design II, Design II has slightly more rebar, which will lead to smaller plastic deformation. However, the required force to re-center the column is also higher due to the increased rebar area.

For other performance groups including relative deck & abutment displacement, and bearing displacement, Design I and Design II also show similar damages, whereas Design III has smaller damages. This also confirmed that the rebar ratio difference between Design I and Design II does not affect EDP significantly. They are more related to the capacity of the structure rather than demand. It is also concluded that when PGA increases, the differences between damages of different designs reduce. It was also noted that the vertical displacement of the bridge was minimal. This is because the self-weight of the structure can counterbalance the vertical excitation. In CHBDC 2014 (CSA, 2014), the design load combinations do not have to include the vertical ground motion if a reduced dead load factor is used. The results in this study also confirmed this method.
Figure 5.8 Maximum drift ratio

Figure 5.9 Residual drift ratio
Figure 5.10 Relative deck & abutment displacement (left)

Figure 5.11 Relative deck & abutment displacement (right)
Figure 5.12 Bearing displacement (left)

Figure 5.13 Bearing displacement (right)
5.5 Repair cost and time

In this study, the repair cost and time calculation is obtained from discrete damage states of performance groups. The social cost such as traffic interruption and delay is not considered. For each damage state, there is a set of prescribed repair quantities and corresponding costs. The repair method used in this study is selected from the benchmark reinforced concrete bridge evaluation (Mackie et al., 2008). The repair method from the benchmark bridges is only suitable for regular bridges. In some situations the bridge projects can be much more complicated, which may involve available human resources, technologies, climatic conditions and even politics. When considering common regular bridges, many parameters can be assumed within a reasonable range. For example, the steel weight per unit volume in the bent cap is 90 kg/m³ and the steel weight in single column bent is 268 kg/m³. In other cases, the repair methods should be proposed by experienced engineers who are familiar with specific structures. In the study of performance based evaluation by Mackie et al. (2008a), the repair methods are developed by researchers and maintenance engineers jointly. The damage states are named from DS0 (i.e. start of a damage) to DS3. A higher number means a more serious damage. Damages below DS0 do not need to be addressed. It corresponds to the Minimal Damage in CHBDC 2014 (CSA, 2014) in which no rebar yielding is occurred.

For the damage of columns, the considered states are concrete cracking (DS0), concrete spalling (DS1), rebar buckling (DS2) and column failure (DS3). For DS0, the repair action is to use epoxy to inject cracks. For DS1 and DS2, the required repair is to remove and patch concrete. From DS2 to DS3, the cost would increase significantly because of
replacement of columns. To replace a column, temporary support for superstructure is needed. A part of column foundation under grade is to be excavated and backfilled. A comparison between the damage states in this section and CHBDC 2014 is shown in Table 5.2. From the table it can be seen that the criteria from CHBDC 2014 is similar to what has been used in this section. In this section a drift of 1.57% is defined as minimal damage, 6.7% drift is defined as repairable buckling and 7.8% drift is defined as extensive damage. However, the repair cost cannot be completely based on the criteria in CHBDC 2014 (CSA, 2014). For example, in the CHBDC 2014 rebar yielding is defined as one of the damage criteria. However, for regular bridges, when the rebar reaches yielding the concrete cover spalling is not occurred. Therefore, it is not likely to detect rebar yielding and perform corresponding repair.

<table>
<thead>
<tr>
<th>Damage states</th>
<th>Drift</th>
<th>CHBDC 2014</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>0.23%</td>
<td>Not Defined</td>
</tr>
<tr>
<td>Spalling</td>
<td>1.57%</td>
<td>Minimal</td>
</tr>
<tr>
<td>Bar Buckling</td>
<td>6.70%</td>
<td>Repairable</td>
</tr>
<tr>
<td>Failure</td>
<td>7.80%</td>
<td>Extensive</td>
</tr>
</tbody>
</table>

For the damage of superstructure, it is assumed that a linear relation exists between the maximum deck concrete strain and the deck surface area. For example, DS0 reaches when 2% of spalling strain is exceeded, DS1 reaches when 25% of spalling strain is exceeded and DS2 reaches when 50% of spalling strain is exceeded. The repair of deck is mainly performed by methacrylate overlay. This repair involves cleaning the deck, methacrylate furnishing and deck treating (Mackie et al., 2008). In terms of bearing failure, Mackie et al. (2008) suggested that only one level of damage needs to be defined.
So the only level of damage is the shear failure. The most efficient way to repair bearings is to replace them because the cost of bearing is small compared to the installation. This simple repair method is applicable to regular bridges but may not be suitable for all projects. Many innovative alternatives are available such as high damping rubber bearings (HDRBs) incorporated with shape memory alloy (SMA) wires (Dezfuli & Alam, 2014) and SMA-based natural rubber bearings (Dezfuli & Alam, 2013). However, in the scope of this study, only traditional retrofit methods are investigated. The common repair method of foundation is to enlarge pile caps and add more piles. For non-structural members such as barriers and lighting pole, their damage states can also be related to structural member EDP such as the maximum column drift. For instance, the damage of lighting pole should be induced by the deflection of the bridge column and the deck. Therefore, the damage of the lighting pole can be estimated based on column damage; however, more research investigation is needed. When the data for non-structural member damage is available, the repair cost estimate should be included.

Estimating bridge repair cost can be a very complex procedure because the repair techniques, cost, and time changes significantly depending on the projects. Also, the repair cost of non-structural members can be difficult to estimate and sometimes it can be more expensive than structural members such as the rail track on a high speed rail bridge. It can also be an amusement purpose Ferris wheel built on a bridge, which is the case in Tianjin, China. In the program Bridge PBEE, the damages are discrete to different states in order to calculate the repair cost and time. For example, minor damage led to column
retrofit and major damage led to column replacement and abutment repairs (Mackie et al., 2010).

In the cost estimate, monetary costs were adjusted to 2015 values based on Caltrans cost index data (Lu et al., 2011). The repair cost quantities of Design I is shown in Figure 5.14. As can be seen from this figure, at PGA values higher than 3.2g, the abutment is damaged and temporary support is needed. As a result, the repair cost has a significant increase. This repair quantity would need much more efforts than other quantity, therefore, the decision may consider other alternatives to minimize the damage if desired so. The repair of abutment involves the excavated and backfilled foundation, temperature support and bridge closure, which can be very expensive. The total repair cost ratio is shown in Figure 5.15. The cost ratio is the ratio of the repair cost to new bridge construction cost. The new bridge construction cost is estimated based on deck area, which is assumed at $ 2484 per square meter (Mackie et al., 2011). Figure 16 shows that Design II requires slightly lower repair cost. This is because repair cost is not only a function of reinforcement ratio. It is also affected by column diameter and other structural parameters.

In such situations, the bridge designers and the owner may consider the smaller rebar ratio (Design I) if the reduced repair cost does not worse the initial effort. To better compare the repair time at different PGA levels, the author categorized the PGA range to 5 zones, which are zone 1 (0 to 0.16g), zone 2 (0.16g to 0.26g), zone 3 (0.26g to 0.5g), zone 4 (0.5g to 0.66g) and zone 5 (0.66g to 1.0g). It should be noted that in low seismic
zones (zone 1), Design I and Design II require similar repair time, Design III generally does not need to be repaired. In low to median seismic zones (zone 2) and median to high seismic zones (zone 4), Design III requires much less repair time than two other designs. Three designs require the same amount of repair time. In the median to and high seismic zones (zone 3 and zone 5), all designs require the same repair cost. Design III requires significantly less repair time comparing with the other two designs. This information could be important for bridge owner in this seismic zone 3 and consider repair time as an essential decision variable. These finding are crucial for bridge owners to make decisions when they consider repair time is of concern. For instance, if at the design level the PGA is in zone 3, the decision makers make need to consider Design III rather than Design I and II. However, if the PGA is within zone 1 and zone 2, it may not be necessary to choose Design III.

![Figure 5.14 Repair cost of different components](image)

Figure 5.14 Repair cost of different components
5.6 Summary

This chapter compares the seismic performance of three bridge designs in terms of engineering demand parameters and decision variables. The engineering demand
parameters include various maximum displacements and residual displacements. The
decision variables are the repair cost and repair time, which are the performance indicator
in the next generation PBD. Three designs (Design I, Design II and Design III) were
considered. The only differences of the three designs were the column diameter and rebar
ratio. Design I and Design II have rebar ratio of 1.9% and 2.7%; the column diameter is
0.914m. Design III has rebar ratio of 5.3% and the column diameter is 1.22m. The
following conclusions can be drawn:

1. Design III shows less seismic vulnerability in all engineering demand parameters
   and decision variables. Design I and Design II show similar engineering demands
   and repair costs after earthquake. The optimized solution of the design would be
   highly dependent on the bridge owner’s judgement. By performing the PBD
   analysis, engineers will be able to provide bridge owners reliable and detailed
   information. So that bridge owners can make the best decision in different
   situations.

2. When comparing the repair time of the three designs, results can be sensitive to
   intensity measurement. In low seismic zones, different designs require similar
   repair time. In low to median seismic zones and high seismic zones, Design III
   requires much less repair time than the other two designs. In median to high
   seismic zones, all three designs require the same repair cost.

3. Residual displacement is a good indicator of bridge damage and fragility. However,
   evaluating seismic performance in terms of economic losses will show the
   benefits and losses more explicitly.
Chapter 6: CONCLUSIONS

6.1 Summary

Canadian Highway Bridge Design Code (CHBDC) 2014 initiated Performance-Based Design (PBD) in Canada. For Lifeline bridges and irregular Major Route bridges, PBD has to be used to explicitly demonstrate structural performance. Regular Major Route bridges can be designed by using FBD or PBD method. In this study, a concrete bent highway bridge is designed using both FBD and PBD based on CHBDC 2014, and FBD based on CHBDC 2006. Soil-structure interaction is incorporated using p-y method in the design and evaluation. Dynamic time-history analyses are performed to assess the seismic performance. The assessment is based on the maximum strain limits from CHBDC 2014. It is found that the prescribed FBD and PBD cannot generate consistent design results to each other.

The displacement based approach has been used to examine the performance criteria in CHBDC 2014. According to CHBDC 2014, the design displacement at 1/475-year event equals to yielding displacement. Therefore, the design displacement based on steel strain of 0.05 at 1/2475-year event divided by yielding displacement is the displacement ductility capacity of the structure. This study introduces the use of ductility demand and capacity to judge which event dominates the design. When ductility capacity from the structure is greater than ductility demand from spectral displacement, a low level design (1/475 year) governs. In general, ductility demands of many Canadian cities range from two to three. This is demonstrated by using displacement spectra from several major
Canadian cities. By comparing the ductility demand and capacity, the PBD process for CHBDC can be greatly simplified.

At the end, the methodology of the next generation PBD is adopted. Three bridge designs are compared in terms of structural damage, repair cost and repair time. Earthquake motions are used to determine intensity measurement, which generate the engineering demand parameter. Then damage measurement is determined by engineering demand. At the end, decision variables are determined by damage measurement.

6.2 Limitations of this study

The study has the following limitations:

1. The research was performed by designing and analyzing ductile substructures. The study of other types of structures may result in different conclusions. The conclusions were drawn from limited case studies. Therefore, the conclusions may not be applicable to other structures if the design background is different.

2. Due to the availability of the data of repair cost and repair time, only single column bridges have been studied while investigating the next generation of PBD.

3. In the current repair cost and repair time estimation, only regular repair techniques have been considered. In the future research, more sophisticated approaches should be considered and this may affect the findings.

4. In the program BridgePBEE, the social cost cannot be considered comprehensively. For example, the cost of interrupting traffic is not well
addressed. Additionally, the unit costs of repair components are not subdivided into very detailed levels.

5. The cost of repair in the study is adopted from California area, therefore the conclusions may not be applicable to other areas that have different characteristics in bridge industry.

6.3 Conclusion

The following conclusions can be drawn from this study

1. Bridges designed as per FBD from CHBDC 2006 and CHBDC 2014 are able to protect life safety. PBD in CHBDC 2014 has very strict requirements compared to CHBDC 2006. The current PBD is also more conservative than FBD in CHBDC 2014.

2. When performing PBD at different earthquake events, it is critical that the soil stiffness degradation be considered. Soil stiffness affects displacement demands significantly.

3. In the cases where steel strain governs the design, the design drifts increase linearly with steel strains.

4. The displacement demand for many cities in Canada at 1/2475-year event is two to three times that of 1/475-year event. However, the ductility capacity of RC column is generally above five. This results in the fact that 1/475-year event tends to govern many ductile structure designs.
5. The application of the next generation of PBD could predict earthquake explicitly, which would be of great value for decision makers.

6.4 Recommendations for future research

The followings are the recommendations for future research:

1. PBD criteria may need to evolve in future practice and research. Currently the CHBDC 2014 does not allow rebar yielding. This may be very challenging to achieve, and may not be the optimized value for the development of regional economic.

2. PBD criteria need to be developed for different types of structures systems, such as unbounded post-tensioned concrete structure, shape memory alloy reinforced concrete structures etc. The current performance criteria are mostly for regular reinforced concrete and steel structures.

3. The next generation PBD is still in its initial stage. To properly estimate the repair cost and repair time, regional data has to be made available. More research on optimizing repair techniques and construction are needed.
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Appendices

Appendix A: p-y curves

The p-y curves for all bridge bents and abutments are shown from Figure A.1 to Figure A.6.

Figure A.1 p-y curves at Abutment 0 from 1m to 9m depth
Figure A.2 p-y curves at Abutment 0 from 10m to 20m depth

Figure A.3 p-y curves at Pier 1 from 0m to 9m depth
Figure A.4 p-y curves at Pier 1 from 10m to 20m depth

Figure A.5 p-y curves at Pier 2 from 0m to 9m depth
Figure A.6 p-y curves at Pier 2 from 10m to 20m depth

Figure A.7 p-y curves at Pier 3 and Abutment 4 from 0m to 9m depth
Figure A.8 p-y curves at Pier 3 and Abutment 4 from 10m to 20m depth