SEISMIC PERFORMANCE EVALUATION OF MULTI COLUMN BRIDGE BENT RETROFITTED WITH DIFFERENT ALTERNATIVES

by

Abu Hena MD Muntasir Billah

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ABSTRACT

Highway bridges constitute a large portion of the national wealth and build up the foundation for economic development. Due to aging and deterioration, they require regular monitoring, evaluation and repair. More than 40% of Canadian bridges have crossed half of their anticipated service life. Many of these are structurally deficient and require major maintenance and rehabilitation. Although budget is allocated each year for maintenance and rehabilitation programme, the amount is usually small and covers only 30% to 70% of the actual maintenance needs. This fact raises the need for identifying bridges that require immediate attention where a significant portion of maintenance resources should be utilized. This study developed an integrated bridge prioritization index for a network of bridges to determine the prioritized work considering the importance of the bridge, cost associated with its rehabilitation and current condition. Once the bridge has been selected it is necessary to select proper retrofit techniques. In order to select a suitable retrofit technique this research has compared the performance of a pre-1965 designed multi column bridge bent retrofitted with different rehabilitation techniques, namely FRP jacketing, steel jacketing, concrete jacketing and Engineered Cementitious Composites (ECC) jacketing. The performance of the four different retrofitting strategies is compared in terms of base shear capacity demand ratio, ductility demand, residual drift and damage states obtained from nonlinear Incremental Dynamic Analysis (*IDA*) and static pushover analysis. Statistical comparisons of static (pushover) against dynamic analyses results have been performed in terms of performance criteria such as displacement and base shear at cracking, yielding and crushing. Moreover, this research assessed the fragility of this retrofitted multi column bridge bent under near fault and far field ground motions. The study aimed to capture the impact of different retrofit techniques on the vulnerability of a retrofitted bridge bent. Through rigorous analyses and applying multi-criteria decision making this study developed a decision making tool that will assist in identifying the most effective retrofitting scheme considering its performance under seismic hazards. The results showed that bridge bent retrofitted with ECC jacketing performed better and deemed to be the optimal retrofit alternative.

PREFACE

Major portions of the work outlined in this thesis have been submitted (see list below) for possible publication in peer reviewed technical journals. The author carried out all analytical work, modeling process, and writing of the initial draft of all papers listed below. The contributions of his research supervisor consisted of providing guidance and supervision, and helping in the development of the final versions of the publications.

Refereed Journal Publications

Billah, A.H.M.M. and Alam, M.S. 2011. Development of a prioritization methodology for maintaining bridge infrastructure systems. Submitted to *Computer-Aided Civil and Infrastructure Engineering.*

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Billah, A.H.M.M. and Alam, M.S. 2011. Fragility analysis of retrofitted multi-column bridge bent subjected to near fault and far field ground motion. Submitted to *Engineering Structures*.

Billah, A.H.M.M. and Alam, M.S. 2011. Performance based multi criteria decision making for seismic retrofitting of bridge bent. Submitted to *Earthquake Spectra*.

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BPI	Bridge Prioritization Index
IFI	Importance Factor Index
CI	Cost Index
BCI	Bridge Condition Index
BHI	Bridge Health Index
ADT	Average Daily Trafiic
DL	Detour Length
RSL	Remaining Service Life
CG	Contribution to GDP
CU	Utility Crossing
EF	Economic Factor
SF	Social Factor
PF	Performance Factor
BL	Bridge Location
SC	Superstructure Condition
CS	Condition of Substructure
DC	Deck Condition
DDC	Driver Delay Cost
VOC	Vehicle Operating Cost
AC	Accident Cost
UC	User Cost
CR	Consistency Ratio
IC	Consistency Index
λ_{max}	Eigen Value
S_a	Spectral Acceleration
IDA	Incremental Dynamic Analysis
SPO	Static Pusover Analysis

PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
T_d	Time Duration
PSDM	Probabilistic Seismic Demand Model
EDP	Engineering Demand Parameter
DS	Damage State
DI	Damage Index
IM	Intensity Measure
SI	Spectral Intensity
λ	Mean
ξ^2	Standard Deviation
PIS	Positive Ideal Solution
NIS	Negative Ideal Solution
PT	Percent Top
AT	Absolute Top

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I have been fortunate to be surrounded by an excellent group of graduate students in the research group who have offered technical knowledge, lively discussions, and friendship. Thanks to my wonderful roommates Nurul, Samy and Rafiqul. I would also like to acknowledge Dr. Anjuman Shahriar for her advice and critical assessment.

I would like to pay my innermost homage to my parents, whom I feel to be the key source of inspiration for all my achievements. Finally, I would like to thank my wife, Sumaiya. Her support, encouragement, and enduring love have meant the world to me throughout this process and always.

DEDICATED TO MY PARENTS

CHAPTER 1 : INTRODUCTION AND THESIS ORGANIZATION

1.1 GENERAL

The highway transportation system is an indispensable component of the modern civil infrastructure and considered as a critical foundation for a country's economic development. They perform as the arteries to establish link between cities and across country to provide a smooth and fast communication system. The failure of bridges during a seismic event not only affects the transport network but also severely impacts the post earthquake emergency response resulting in severe economic losses. The transportation system of North America is particularly at risk because of the history of large but infrequent events and the fact that many of its bridges were designed with little or no seismic consideration.

Over the last two decades Canadian highway system has continued to deteriorate. In particular, the deteriorated bridges are the most crucial and vulnerable components of the transportation network. Being caught in a fiscal squeeze caused by low funding, population growth, environmental requirements, poor quality control leading to inferior installation, and inadequate inspection and maintenance, have severely affected the highway system. Although the investment on highway system has an upward trend, it is not sufficient to meet the annual rehabilitation needs for the existing bridges, or alleviate the backlog of maintenance and rehabilitation that accumulated over the decade (Mirza, 2007). This fact raises the need for identifying bridges that require immediate attention where the maximum portion of maintenance resources should be utilized. Identifying particular bridges for such operations is a daunting task as it involves numerous inter-related decision parameters and it has become increasingly important for decision-makers to use decision tools to make proper investment decisions.

Bridge bents are one of the most vulnerable elements in a bridge whose failure can have catastrophic consequences. Bridge bents are reinforced concrete (RC) frames commonly used to support beams and girders. In recent years, awareness of the potential seismic hazard of bridges constructed before 1970s has increased, and has accelerated seismic retrofit activities all around the world. However, this has led to a question of the suitability of different retrofit techniques available for bridges. Little analytical and experimental investigation has been offered to date for evaluating the impact of various retrofit measures on the seismic performance of multi-column bridge bents. There is a strong need for a comparative assessment of the viability of various retrofit strategies for multi-column bridge bents.

Development of fragility curves can facilitate decision making for seismic retrofit of multi-column bridge bent. It is indispensable to evaluate the seismic vulnerability of bridges against earthquakes. The seismic vulnerability of highway bridges is often explicitly expressed in the form of fragility curves. Fragility curves indicate the conditional probability of a bridge sustaining a particular degree of damage when subject to a given level of ground shaking. The availability of reliable retrofitted bridge bent fragility curves would allow for assessment of the various retrofit measures on the performance of retrofitted multi-column bridge bent.

A proper selection of suitable retrofit technique is a very important issue as an improper selection can have unintended consequences on the overall seismic performance of a bridge system. As different performance indices are considered for performance-based design and assessment of structures, it becomes a crucial problem for the engineers to select suitable retrofit strategy that meet a set of predefined limits corresponding to a desired performance level (*PL*). This performance evaluation and optimal selection of retrofit alternatives have multi-level and multi-factor features and can therefore be regarded as multiple criteria decision-making (MCDM) problem. MCDM can be a viable solution for selecting suitable retrofit technique from a set of available alternatives.

1.2 OBJECTIVES OF THE STUDY

The key objectives of the current research include:

1. Develop and exercise a methodology for prioritizing bridge structures and develop a bridge prioritization index that will assist decision makers in the resource allocation of funds for bridge infrastructure upgrading.
2. Determine the seismic performance of a multi-column bridge bent retrofitted with different alternatives and compare their performance in terms of base shear capacity demand ratio, ductility demand, residual drift and damage states. Carry out statistical comparisons of static (pushover) against dynamic results, in terms of performance criteria such as displacement and base shear at cracking, yielding and crushing.
3. Assess the seismic vulnerability of the retrofitted bridge bents under near fault and far field ground motion.
4. Develop a decision support tool using multi criteria decision making to identify the most effective retrofitting scheme considering its performance under seismic hazards.

1.3 SCOPE OF THE RESEARCH

In order to achieve the goals of the study, the existing methodologies to identify and prioritize bridges were obtained from the literature review. The study presents the state-of-the-art and current methods used to prioritize bridges and distribute maintenance funds. A multi column bridge bent built and designed prior to 1965 has been retrofitted with different alternatives and their performance was evaluated under seismic excitations. The procedures to achieve the stated objectives of the study are stated as follows:

1. Developing an integrated bridge prioritization index that emphasizes the importance of bridge infrastructure to the overall economy, performance level of bridges, social importance of bridges, etc., for simulating maintenance priority orders. The developed method utilizes a knowledge based evaluation technique by considering location and topology of the bridge, contribution to GDP, traffic volume, alternate route, and bridge condition.
2. Analytical 2D models were generated using software SeismoStruct (2010) for retrofitted multi-column bridge bents. Several retrofitting techniques specifically concrete jacketing, steel jacketing, Carbon Fiber Reinforced Polymer (CFPR) jacketing and Engineered Cementitious Composites (ECC) jacketing have been considered in this study to improve the seismic performance of a gravity load designed bridge bent under seismic forces.
3. The performance of the four different retrofitting strategies is compared in terms of the base shear capacity demand ratio, ductility demand, residual drift and damage states obtained from static pushover analysis (*SPO*) and incremental dynamic analysis (*IDA*). Statistical analysis have been performed to compare the results obtained from *SPO* and *IDA* in terms of performance criteria such as displacement and base shear at cracking, yielding and crushing.
4. The seismic vulnerability of the retrofitted bridge bents were evaluated by developing fragility curves under near and far field ground motions. Fragility curves were developed

using Probabilistic Seismic Demand Models (*PSDM*). Displacement ductility is considered as the demand parameter and Peak Ground Acceleration (*PGA*) is taken as the intensity measure (*IM*) to develop fragility curves.

5. The best retrofit alternative for seismic retrofitting was determined based on their performance under seismic excitation by applying TOPSIS multi-criteria decision making method. The performance criteria used in this study are base shear capacity demand ratio, residual displacement, ductility capacity and energy dissipation capacity.

1.4 THESIS ORGANIZATION

This thesis is arranged in seven chapters. In the present chapter a short preface and the objectives and scope are presented. The content of the dissertation is organized into the following chapters:

In **Chapter 2**, a comprehensive literature review on common seismic deficiencies found in existing bridges along with a detailed review of researches related to the application of bridge retrofit and strengthening techniques is presented. This chapter also examines the application of various retrofitting techniques developed for bridge strengthening and their comparative performance evaluation.

In **Chapter 3**, a prioritization methodology has been developed for maintaining bridge infrastructure systems. The developed methodology will identify bridges that will require rehabilitation/retrofitting through screening processes considering their importance on the road network, cost associated with its rehabilitation and current condition. The proposed prioritization technique is expected to play a significant role in assisting decision makers to provide a rational ranking among candidate bridges and thus, detect the critical ones for fund allocation.

Chapter 4 demonstrates the seismic performance of a multi-column bridge bent retrofitted with different alternatives specifically concrete jacketing, steel jacketing, CFRP jacketing and ECC jacketing. To compare the performance of various retrofit techniques static pushover analysis were performed, followed by the nonlinear incremental dynamic time history analyses. The results obtained from *IDA* and *SPO* are compared in terms of displacement and base shear at cracking, yielding and crushing by conducting statistical analyses

Chapter 5 demonstrates the seismic vulnerability assessment of the retrofitted bridge bents under near fault and far field ground motions by developing fragility curves. Fragility functions are derived based on simulation results from nonlinear time history analyses, and then they are combined to evaluate the overall fragility of the bridge bents. *PSDM* is employed here to derive the analytical fragility curves using cloud approach. In developing the fragility curves displacement ductility of bridge pier is considered as the *EDP*, and the *PGA* is utilized as *IM* for each ground motion record.

Chapter 6 proposes a simplified and systematic approach for selecting a suitable seismic retrofit technique based on their seismic performance. Retrofit selection is a MCDM problem where many conflicting criteria need to be considered in decision making. This study focuses only on the performance criteria as safety is the prime concern for vital facilities such as bridges. The performances have been evaluated by performing non linear static and dynamic analyses techniques. Applying MCDM, ECC jacket was found to be the most suitable retrofit option.

Finally, **Chapter 7** presents the summary and conclusions attained from this research program. Few specific recommendations for future research have also been suggested.

CHAPTER 2: LITERATURE REVIEW

2.1 GENERAL

Bridges constructed before 1970 underestimated the effect of seismic forces in many regions. The behaviour of reinforced concrete (RC) structures under dynamic loading with reversed cycles was often ignored during design procedure. As a result older bridges offer insufficient resistance to lateral excitation and fail to provide satisfactory performance in a major seismic event. Consequently, these bridges are under the threat of experiencing severe damages even during a moderate earthquake. The 1971 San Fernando earthquake is often described as a remarkable event in bridge engineering as it demonstrated the poor design practice of that period (FHWA-HRT-06-032, 2006). This accelerated the need for seismic assessment of these poorly designed bridges and then developing retrofit schemes to upgrade their seismic performance that matches current code requirements. Several methods for upgrading these seismically deficient bridges have been developed over the years. This retrofitting process involves improving the performance of various elements such as columns, footings, cap beams, knee joints, and abutments. Bridge failure during a seismic event may be attributed to the deficiencies in any of these structural components. After the catastrophic effect of 1989 Loma Prieta earthquake, a major effort was undertaken to perform comprehensive seismic retrofits on a large number of bridges. Initially, this effort was focused on bridges with single column piers, which were believed to be the most vulnerable to collapse. However, many bridges with multi-column piers collapsed or were severely damaged during the 1994 Northridge earthquake, and bridges of this type were subsequently added to the CALTRANS retrofit program (Buckle, 1994).

Countries such as USA, Japan, Canada and New Zealand have carried out extensive seismic retrofitting of highway bridges over the last few decades. Due to these parallel efforts, significant progress has been made in the state-of-the-art of retrofitting bridge superstructures, columns, and foundations. Several retrofitting techniques such as RC jacketing and steel jacketing have been developed to rehabilitate structurally-deficient bridge columns. In the last two decades, fiber reinforced polymers (FRP) have attracted the attention of researchers and bridge owners as an alternative material for retrofitting RC bridge elements. Other commonly used retrofitting method includes external hoops tensioned by turnbuckles, active confinement by wrapping with prestressed wire.

Illinois Institute of Technology is known as the pioneer to conduct a research project dealing with the retrofitting of bridges (Robinson et al., 1979). In 1981 Federal Highway administration published the first guidelines for retrofitting highway bridges in a report titled *Seismic Design Guidelines for Highway Bridges*, FHWA-RD-81-081. Following this Applied Technology Council developed guidelines for seismic retrofitting of highway bridges (ATC-12). This guideline provided formal screening and evaluation procedure to identify bridges that require retrofitting and presented retrofit concepts that in many cases had not been used in practice at that time. Following the Loma Prieta earthquake CALTRANS conducted extensive seismic assessment and retrofitting program on its huge inventory of over 15000 bridges (Pristely and Seible, 1991). Since then, researchers have conducted extensive investigations that reflected the advancement in the practice of seismic retrofitting, which includes improved methods for restrainer design (Randall et al., 1999; DesRoches and Fenves, 1998); methods for improving the performance of older steel bridge bearings (Mander et al., 1998a); improved methods of analysis and retrofit of RC columns (Dutta et al., 1999); retrofit methods for multicolumn reinforced concrete piers (Mander et al., 1996a and b).

This chapter provides a comprehensive literature review on common seismic deficiencies found in existing bridges along with a detailed review of researches related to the application bridge retrofit and strengthening techniques.

2.2 COMMON SEISMIC DEFICIENCIES OF EXISTING BRIDGES

Researchers have carried out extensive studies to report the deficiencies exist in bridges and damages occur during major earthquakes. These include 1971 San Fernando earthquake (Housner, 1971), 1989 Loma Prieta (Bruneau, 1990; Mitchell et al., 1991), 1994 Northridge (Mitchell et al.,1995; Seible and Priestely, 1999) and 1995 Kobe earthquake (Anderson et al., 1996; Taylor, 1999). A study conducted by Mitchell et al. (1994) categorized some common deficiencies prevailing in older bridges. Tables 2.1-2.3 describe the common deficiencies found in older bridges.

Table 2.1: Deficiencies observed in columns of existing bridges

Deficiency	**Consequences**
Large tie spacing/ large pitch on spirals.	Insufficient confinement of core concrete. Unable to prevent longitudinal bar buckling. Susceptibility to shear failure.
Splicing of longitudinal bar at column base	Limited hinging length. Inadequate splice length unable to transfer the full tensile force of the longitudinal reinforcement to the starter bars of the foundation.
Absence of intermediate cross ties	Inadequate shear strength. Inadequate control of bar buckling.
Large spacing of transverse and longitudinal bars	Poor column confinement.

Table 2.2: Deficiencies observed in cap beams of existing bridges

Deficiency	Consequences
Insufficient anchorage length of the longitudinal reinforcement	Low positive moment capacity. Insufficient flexural strength.
Inadequate shear reinforcement	Insufficient shear strength.
Inadequate embedment of bottom bars in joints	Insufficient development of flexural reinforcement.

Table 2.3: Deficiencies observed in bridge bent joints of existing bridges

Deficiency	Consequences
Lack of vertical and horizontal shear reinforcement	Insufficient shear strength.
Large diameter bends necessary for large bar sizes	Reduction of the effective depth of the joint region. Formation of a weak shear plane through the joint.
Poor anchorage details of transverse and flexural reinforcement	Insufficient shear and flexural strength of joints.

2.3 PREVIOUS RESEARCH ON RETROFITTING OF BRIDGE BENTS

Researchers have carried out numerous experimental and analytical studies on seismic retrofit of deficient bridges. A significant portion of the initial research provided insight into the effectiveness of different retrofit measures to improve both flexural and shear strength, and flexural ductility of reinforced concrete bridge columns (Chai et al., 1992). When a bridge structure is subjected to lateral excitation, failure can be caused by failure of the supporting soils and foundation system, or by failure of the lateral force resisting elements of the structure itself. The column bent/ bent cap provides one such lateral force resisting system in the moment resisting frame of a bridge. Figure 2.1 shows a typical multiple column bridge bent and its components. Multi column bents and cap beams are widely used for highway bridges. Extensive damage can take place due to deficiencies present in those columns, cap beams and column-cap beam joints.

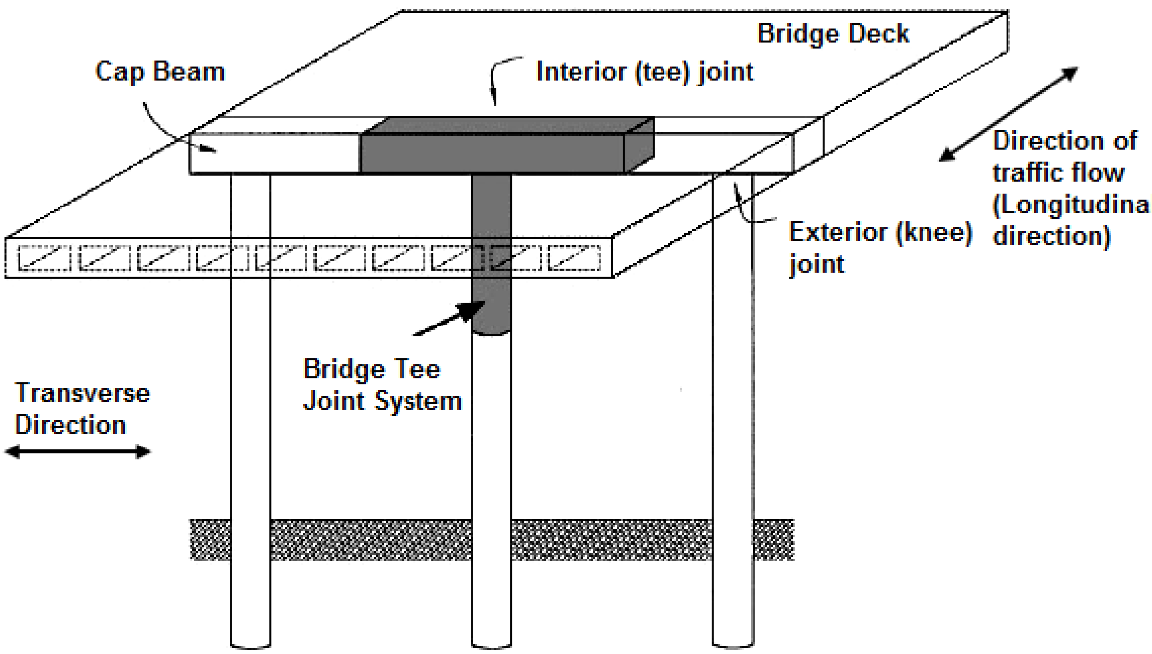


Figure 2.1: Multiple column bridge bent

Cap beams act as the link to transfer forces between the superstructure and columns. Cap beams usually fail in flexure and shear, particularly in outrigger bent caps. Low positive reinforcement ratio in column faces, and premature termination of negative reinforcement are the common deficiencies found in older multi-column bridge bents. The deficiencies found in cap beams are critical and difficult to rehabilitate. As the older cap beams and joints were not detailed to behave in a ductile manner, the retrofit design must ensure that these elements are either capable of accommodating the ductility demands placed on them, or are capable of elastically resisting the forces that will result from plastic hinging in the columns.

Retrofitting of beam-column joints in RC bridges were carried out by Lowes and Mohele (1999). They applied two different methods for retrofitting beam-column T-joints in RC bridge structures. In the first method, they added RC bolsters to the cap beam and joint. In the second method, they used a post-tensioned concrete retrofit connection involving addition of posttensioned concrete bolsters to the cap beam and joint. The first method displayed moderate ductility capacity while the second one displayed large ductility under simulated earthquake and gravity loads. The additional post tensioning in the second method improved both joint shear strength and anchorage strength of column longitudinal reinforcement. Priestley et al. (1993a, 1993b) developed a retrofit concept by prestressing the cap beam in order to improve flexural and shear strength of the existing cap beam and edge beams. They conducted experimental investigation on large scale models and reported the dependability and conservativeness of these approaches for flexure and shear capacity enhancement. Mitchell (2002) strengthened RC bridge bent both in shear and flexure by adding reinforced concrete sleeve in beam. The beam sleeve was constructed using high performance, steel fibre reinforced concrete. They placed a steel jacket filled with concrete grout around the column to increase the confinement and shear reinforcement. They tested the specimen under reverse cyclic loading. The retrofitted bent exhibited a stable hysteretic response with no significant loss in strength with good energy dissipation capacity. Griezic (1996) tested a quarter scale outrigger beam-column joint and five half scale RC bridge columns, which had typical details of bridges designed and constructed during 1960. He retrofitted the specimens with addition of reinforced concrete to increase the shear resistance and tested under reverse cyclic loading. The specimens demonstrated increased strength, ductility and energy dissipation capacity. Halling et al. (2001) retrofitted T-joints and bridge pier using CFRP composite jacket. They carried out full scale tests and compared the results with the analytical findings. Test results demonstrated that after retrofitting, ductility of each joint increased significantly up to 50% while the increase in strength was only 5%. Ingham et al. (1994) carried out experimental studies on 1/3 scale models of outrigger knee joints by adding reinforcement in the joints to improve the shear capacity. They applied reverse cyclic loading and found better response with increased strength and ductility. Stojadinovic and Thewalt (1995) used concrete jackets for strengthening the beams and confining joints of an existing outrigger knee joint with additional steel plates bolted through the cap beam and joint region. They tested half scale specimens under transverse and longitudinal loading and reported the improved ductility and energy dissipation of the systems. Gergely et al. (1998, 2000) retrofitted bent cap, columns and bent cap column joints of an existing RC bridge in Salt Lake City using CFRP composite jackets. They found that this repair system significantly improved the shear capacity of the column-cap beam joints and ductility of bridge piers. Sritharan et al. (1999) investigated the enhanced seismic performance of bridge cap beam-column joints by prestressing cap beams. They designed and tested cap beams with prestressing under simulated earthquake loading and reported the superiority of joints incorporating prestressing.

2.4 SEISMIC RETROFITTING TECHNIQUES

Seismic retrofitting of existing bridges is considered to be more challenging than construction of a new bridge because of several factors associated with retrofitting. For instance, before retrofitting the seismic performance level and the goals have to be clearly defined. Priestley et al. (1996) presented various seismic rehabilitation techniques of reinforced concrete bridge column using steel, concrete, and fiber-reinforced polymer (FRP) composite jacketing. Engindeniz et al. (2005) carried out extensive literature review on the state-of-the-art repair and strengthening techniques for reinforced concrete beam-column joints. This paper focused on retrofitting RC joints with an attention to actual bridges as well. This section provides a comprehensive literature review on seismic retrofitting techniques available for bridges.

2.4.1 Concrete Jacketing

Concrete jacketing had been the method of choice for rehabilitation of deficient structures. It is generally cheaper than other retrofit measures and it is also a suitable method for retrofitting columns in water. If applied with appropriate reinforcement, concrete jacket can enhance the stiffness, flexural and shear strength as well as the deformation capacity and are effective in achieving enhanced confinement. Concrete jacket has some disadvantages as compared to FRP and steel jacket as it increases the size of the structural members. Figure 2.2 illustrates the concrete jacketing technique. Rodriguez and Park (1975) conducted experimental investigation on rectangular RC column retrofitted with concrete jacketing under simulated seismic loading. They investigated the increase in strength, stiffness and ductility as a result of a damaged and undamaged column encased by concrete jacketing and reported their effectiveness. Concrete jacketing techniques for both circular and rectangular columns have been described in details by Priestely et al. (1996). They reported that construction and effectiveness of concrete jacket is convenient for circular column by using closely spaced hoops or small pitched spirals. Bousias et al. (2006) conducted cyclic loading tests on rectangular column retrofitted with concrete jacketing. They observed that the concrete jacket is effective for retrofitting columns with a lap splice length of, as short as, 15-bar diameter. Lampropoulos and Dritsos (2010) analytically investigated the effect of concrete shrinkage on RC columns retrofitted with concrete jacketing. They found that the shrinkage of concrete jacket reduces the strength of the column and this effect must be considered as it induces slip at the interface between the old and the new concrete and tensile stresses in the jacket concrete.

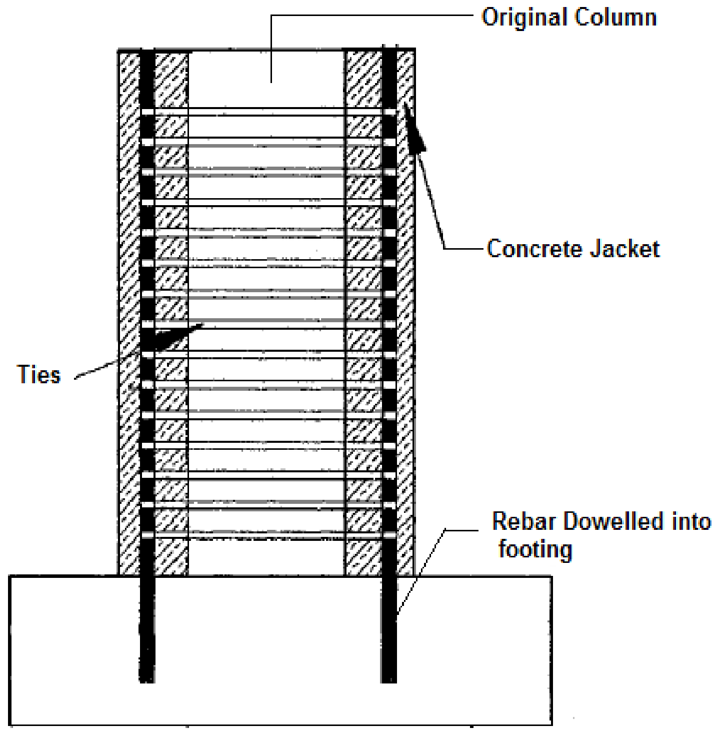


Figure 2.2: Column retrofitting using concrete jacket

2.4.2 Steel Jacket

Columns with insufficient ductility and strength capacity can be upgraded by confining them with steel jackets. The procedure was originally developed for circular columns as shown in Figure 2.3a. For rectangular columns, generally elliptical steel jackets are used (Figure 2.3b). Chai et al. (1991) developed the steel jacketing technique for retrofitting of seismically deficient RC column. They demonstrated that columns retrofitted with steel jacket exhibited ductility similar to that of columns designed following current seismic codes. Based on satisfactory laboratory results (Chai et al. 1991 and Priestely and Seible, 1991), steel jackets have been employed to retrofit both circular and rectangular columns around the world. Tsai and Lin (2001, 2002) proposed octagonal shaped steel jacket for rectangular reinforced concrete column. Their test results indicate that proportioned octagonal steel jackets improve the ductility and cyclic strength of bridge columns that are lacking in flexural and shear strength.

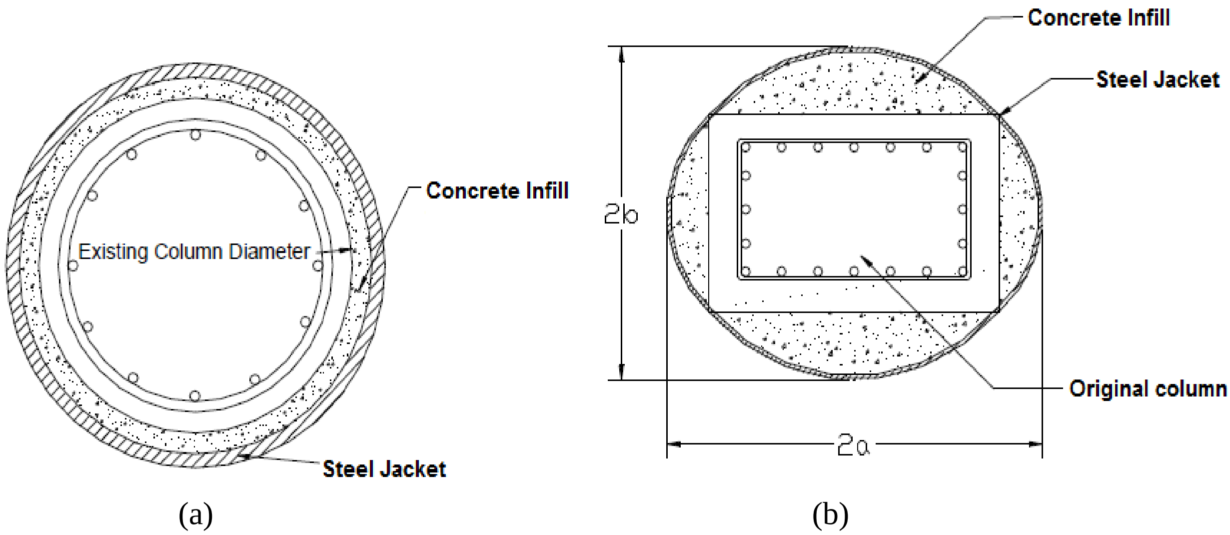


Figure 2.3: Column retrofitting using steel jacket (a) circular column, (b) rectangular column (After Priestely et al, 1996)

After the 1982 Urakawa-oki earthquake, Japan conducted extensive retrofit program of bridge piers using steel jacketing. A series of experimental investigation was carried out (Kawashima, 1990) to retrofit a total of 50 columns. Some of the retrofitted columns were exposed to severe ground shaking during the 1995 Kobe earthquake but none of them suffered any damage. Chai et al. (1996) investigated two aspects of steel jacketing. They investigated the improvement in the lateral stiffness and ductility brought about by the use of steel jacket and the damage of steel jacketed columns during the 1989 Loma Prieta earthquake. They reported that steel jacket provide the required confinement to the concrete column and through dilation process enhance the ultimate compressive strain of concrete. They also concluded that column lateral stiffness is affected by the jacket thickness and length. Zhang et al. (1999) analytically evaluated various combinations of column jacketing of a multi-column bent. They adopted partial and full retrofitting using steel jacketing. They concluded that both measures increased the ductility capacity but the levels of effectiveness were not the same. Daudey and Filiatrault (2000) conducted both experimental and numerical analysis for steel jacketed reinforced concrete bridge piers with complex cross-sectional geometries and lap-splices in plastic hinge region. They came up with the conclusion that a gap of about 50mm is required between the jacket and footing top to avoid stress concentration and premature bar failure. Xiao and Wu (2003) retrofitted both rectangular and circular RC columns using thin steel jackets, which were further stiffened using thick plates or angle iron in the plastic hinge region. They concluded that the thin jacket provided enough shear strength but limited ductility and eventually the column failed as a result of the jacket bulging at the end of the column. Few studies have demonstrated that rectangular steel jackets are not effective enough to extract the full benefit of retrofitting (Sun et al., 1993 and Seible et al., 1990). Although rectangular jackets can increase the shear and moment capacity of rectangular column, it fails to provide sufficient lateral confinement (FHWA-HRT-06-032, 2006, Priestely et al., 1996). Experimental investigations conducted in Japan showed that rectangular jackets can be effective for rectangular column if they are confined by stiffened lateral beam at the bottom (Iwata et al., 2001). Tsai and Lin (2001), Harries et al. (1999), Sun et al. (1993) showed that the rectangular steel jacket for rectangular reinforced concrete column cannot efficiently provide lateral confinement. Extensive testing of single columns with steel jackets was performed by Priestley et al. (1994 a, b) and Chai et al. (1994). They recommended not to use rectangular steel jackets for rectangular column although they are capable of enhancing shear strength.

2.4.3 FRP Composite Jackets

A number of advanced fibre composites namely carbon fibre, aramid fibre and glass fibre composites have been developed to improve the flexural ductility and shear strength, and to correct lap splice length deficiencies at the plastic hinge regions of bridge columns. FRP composite materials possess several advantages over steel and concrete jacketing such as, extremely high strength-to-weight ratios, high elastic moduli, resistance to corrosion, and ease of application. Such properties make FRP composites a suitable candidate for bridge retrofitting. Figure 2.4 shows an example of Interstate 94 Highway bridge at Chesterton, Indiana retrofitted with FRP wrapping (QuakeWrap). The seismic retrofit design guidelines for reinforced concrete column with FRP jacket was proposed by Seible et al. (1997). Experimental and analytical study on circular RC bridge columns strengthened using prefabricated composite jacket was conducted by Xiao and Ma (1997). Their study demonstrated that composite jacket was effective in improving the seismic performance of columns. Xiao and Ma (1999) retrofitted and repaired circular RC bridge columns having poor lap splice details with continuous and segmented prefabricated glass fibre reinforced polymer shell. They reported the effectiveness of using continuous and segmented composite jacket for retrofitting. Nanni et al. (1999) strengthened the piers of an RC bridge using near–surface mounted carbon FRP rods as well as jackets made of continuous FRP sheets and tested the pier up to failure. Saadatmanesh et al. (1997) retrofitted rectangular bridge columns using both rectangular and oval shaped composite jacket and tested under reverse cyclic loading. Their result proved the efficacy of both method in improving ductility and energy dissipation capacity. Haroun and Elsanadedy (2005) retrofitted RC bridge column with poor lap splice detailing using FRP composites. They tested half scale circular and rectangular column under lateral cyclic loading and demonstrated significant improvement in column ductility without considerable stiffness amplification. They found composite jackets did not affect the lateral stiffness of columns, thereby maintaining the bridge dynamic properties. Endeshaw (2008) conducted experimental investigation on 0.4 scale rectangular RC bridge columns wrapped with CFRP. The specimens were subjected to increasing levels of cycled lateral displacements under constant axial load. Experimental results demonstrated that columns retrofitted with rectangular-shaped CFRP jackets showed ductile column performance and failure occurred due to flexural hinging in the gap region followed by low-cycle fatigue fracture of the reinforcement.

Figure 2.4: Bridge bent retrofitted with FRP wrapping (Quakewrap)

Roy et al. (2010) carried out pseudo dynamic tests on a CFRP retrofitted bridge bent to evaluate its performance. The bridge bent was subjected to increasing level of seismic loading, corresponding to various limit states of the bridge. Their results showed that CFRP jacketing performs well under increasing level of dynamic loading while encountering very low strain on the fibers. Application of composite jacketing largely depends on the properties of the composite material such as susceptibility to moisture (Steckel et al., 1998). Glass fibers can lose significant strength due to moisture absorption. As CFRP is less affected by moisture Washington Department of Transportation recommend the use of CFRP for bridges in Washington (WSDOT M 23-60, 2006). These types of problems can be addressed by careful testing of prospective systems under extreme or accelerated service conditions, good quality control during construction and application in the field, and design with sufficient safety factors to account for prospective strength loss. Detailed guidance on the use of composites for column wraps is available in ACI's Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI, 2002). Large scale application of CFRP composite for seismic retrofit of highway bridges was implemented at Sakawa-gawa Bridge, Japan. The bridge was 5 span continuous steel girder bridge supported on 42-65 m hollow circular piers. In an attempt to assert the effectiveness of selected retrofit technique a series of cyclic loading test was carried out on one-20th scale models (Ogata and Osada, 2000). Test result demonstrated the increase in flexural capacity from 3% drift (as built) to over 5% drift (retrofitted). Okamoto et al. (1994) demonstrated the effectiveness of aramid fibre composite jacketing for strengthening of RC column. They tested a shear deficient column of 625mm height and 250mm wide, retrofitted with both aramid braided tape and unidirectional tape under cyclic loading. The as built column failed in shear while the retrofitted column failed in flexure. Kato et al. (2001) demonstrated another practical application of aramid fibre jacket by retrofitting 8-12.5m tall circular bridge piers. Application of aramid fibre jacket increased the flexural capacity of the piers significantly.

2.4.4 ECC Jacketing

In an attempt to change the brittle nature of ordinary concrete researchers have developed modern concepts of ultra high performance fibre reinforced cementitious composites, which are characterized by tensile strain-hardening after first cracking. Engineered Cementitious Composites (ECC) is developed based on micromechanical principles which allow it to strain harden in tension. This principle requires minimum amount of reinforcing fibres (less than 2% by volume) while representing high performance with extreme ductility (Kesner and Billington, 2005). ECC possesses characteristics analogous to a ductile metal which strain hardens after first cracking and exhibits 500 times higher strain capacity than normal concrete. High tensile ductility and the unique crack development make ECC a durable material. Damage tolerance is an important factor in performance-based engineering. High damage tolerance with much finer cracking makes ECC superior than ordinary concrete. When used in structural applications with steel reinforcing, this fine cracking can reduce the ingress of damaging chemicals and moisture. In addition, there are aesthetic benefits to fine cracking as opposed to having larger cracks. The improved tensile properties and damage-tolerant characteristics of ECC make its use appealing for many structural applications. Examples include RC coupling beams (Canbolat et al., 2005 and Yun et al., 2005) and beam-column connections (Parra-Montesinos and Wight, 2000). In terms of utilizing the energy-dissipation characteristics of ECCs, their use has been proposed to provide energy dissipation to systems that do not possess such characteristics, such as frames or members reinforced with FRP bars that do not yield (Fischer and Li, 2003a,b) and for retrofit applications for seismically deficient structures (Kesner and Billington, 2005). Li et al. (2000) experimentally demonstrated the effectiveness of ECC as retrofitting materials for energy absorbing structural shear wall for seismic retrofit of open frame RC buildings. Kabele et al. (1997) numerically investigated the application of ECC in precast shear panels for building wall retrofits. They concluded that the ability of the ECC to relax the stress and redistribute the damage is responsible for the improved structural strength and ductility observed in the panels strengthened by ECC.

2.4.5 External Prestressing Steel

Wrapping prestressing wire under tension around a column can provide an improved form of confinement. This procedure has been reported to successfully increase the flexural ductility of circular columns with lap splices at the critical section, but its effect on shear strength has not yet been quantified. Figure 2.5 shows the application of prestressing steel for column retrofitting. An advantage of this technique is that it has little effect on the flexural strength and stiffness of the column. Reliable anchorage of the wire ends is essential for an effective field application. Coffman et al. (1991) demonstrated the application of prestressed steel hoops to retrofit circular bridge columns. They concluded that this retrofitting technique is effective for long columns where shear is not a significant failure mode. Hawkins et al., (1999) conducted experimental investigation on columns retrofitted with prestressing strand. The results demonstrated columns lose strength at dilation strains beyond 0.001 as a result of high prestress losses that occur due to stressing the strands around a typical column. Swanson (1999) developed a new technique of stressing where a hand-held machine is used to stress the unstressed strand around the column. This stressing technique proved advantageous as it minimizes the prestress losses due to friction.

2.4.6 Precast Concrete Segment Jacket

As steel jacketing is susceptible to corrosion and concrete jacketing requires longer construction period, precast concrete segment jackets are now widely used to retrofit columns in water. This method results in speedy construction compared to the standard reinforced concrete jacketing. Use of prefabricated concrete segments significantly reduces the construction period. Special joints for connection of segments are sometimes used to further reduce construction period. This method is cost effective. Fabrication of segments at the factory and the setting of segments at site reduce the cost as compared to concrete jacketing. The application of precast concrete segment jacket started in Japan after the 1995 Kobe earthquake. The efficacy of this technique was proved experimentally by conducting cyclic loading testing on half scale models. This retrofitting method effectively enhanced the strength and ductility capacity of column (FIB Buletin-39, 2007). One major drawback of this technique is that it cannot be applied to bridges which are supported on columns of unequal height.

Figure 2.5: Column retrofitting using external prestressing (FHWA-HRT-06-032, 2006)

2.4.7 Ferrocement Jacketing

Ferrocement is a thin-walled composite material, where finely divided wire meshes are distributed spatially in the mortar matrix (Naaman 1999). This material shows isotropic behaviour in two principal directions in its plane. Small diameter wires are closely and uniformly distributed over the entire volume of mortar. Ferrocement is a relatively low cost material and does not require any advanced technique for application. Takiguchi and Abdullah (2000) conducted an experimental investigation on the use of ferrocement jackets reinforced with wire meshes for strengthening small square RC column. Test results demonstrated the efficacy of ferrocement jacket in improving the shear strength and ductility of square columns. Kazemi and Morshed (2005) retrofitted shear deficient short concrete columns using ferrocement jacket reinforced with expanded steel mesh. They concluded that ferrocement layer increased the ductility capacity where expanded steel meshes were more effective than ties. An experimental investigation on RC bridge piers strengthened by ferrocement was carried out by Kumar et al. (2005). They tested the specimens under simulated seismic loading and constant axial load. Observation from experimental study revealed the enhanced stiffness, strength, energy dissipation and ductility of ferrocement-jacketed specimens and the mode of failure changed from brittle shear failure to a ductile flexural failure.

2.4.8 Infill Shear Walls

Infill shear walls are walls cast in place often used in retrofitting of multi-column bents. They have been reported to provide high shear and flexural capacity in the transverse direction. These walls prevent the formation of plastic hinges in the columns during transverse loading, and help overcome deficiencies in the flexural or shear strength of the pier cap. Kawashima et al. (1994) retrofitted an 8.7 m tall RC frame pier using Infill shear wall. To be effective, infill shear walls should be designed to act compositely with the existing members. This is usually done by providing a sufficient number of drilled and bonded dowels in the columns and bottom of the pier cap, so that shear is transferred at the interfaces through a shear friction mechanism.

2.5 COMPARATIVE EVALUATION OF RETROFIT TECHNIQUES

This section provides a comparative evaluation of different retrofitting techniques. Among all the techniques discussed above, steel jacket is most widely used all around the world. This is the preferred method used by CALTRANS and they have retrofitted a good number of bridges utilizing this method. Concrete jacket is the oldest form of retrofitting technique. Similar to the steel jacket, concrete jacketing is more effective for circular column. Different combinations of constituent materials, fibre structures and method of application have made FRP composites an attractive alternative for seismic retrofitting. Advantages and disadvantages of various retrofitting techniques are presented in Tables 2.4-2.6.

Table 2.4: Advantage and disadvantage of concrete based jacketing

Technique	Advantage	Disadvantage
Concrete jacket	Increased flexural strength and stiffness, durable, low cost, suitable for application in water.	Additional tensile stress in the jacket concrete significantly reduces column compressive strength as a result of a biaxial stress state.
Ferro-cement Jacket	Low material cost, does not require skilled workmanship, sufficiently flexible to wrap circular or rectangular column.	Low durability, poor aesthetics.
ECC Jacket	High damage tolerance, ductility, strain hardening property, high energy dissipation.	High material cost, limited availability of design guideline, lack of experience.

Table 2.5: Advantage and disadvantage of FRP based jacketing

Technique	Advantage	Disadvantage
Advanced composite	Strong, lightweight, durable, low labour cost.	High material cost, temperature dependent, prone to moisture.
Carbon fibre	Easy to handle and apply, effective in shear and flexural retrofit.	High material cost.
Glass fibre	Enhances ductility significantly, moderate cost.	Susceptibility to absorbed moisture.
Aramid fibre	Light and easy to wrap without heavy machines, good performance under cyclic loading, suitable for columns with variable sections.	High cost, can lose significant strength due to moisture absorption.

Table 2.6: Advantage and disadvantage of steel based jacketing

Technique	Advantage	Disadvantage
Steel based	Readily available, low material cost, good durability.	Corrosion, material weight
Circular jacket	Proven technology, practical application around the world.	High installation cost, difficult to weld, materials handling.
Eliptical jacket	Suitable for rectangular column, provides adequate confinement.	Limited application, high installation cost, special forms required, bracing of jacket during construction.
Plates and angles	Standard steel shapes, readily available, easy to handle.	Huge amount of weld required, quality assurance, high installation cost.
Jacket and stiffeners	Good aesthetics, readily available, easy to handle.	Cutting and welding of stiffeners difficult.
Prestressed external hoops	Little effect on the flexural strength and stiffness of the column.	Reliable anchorage of the wire ends is essential, applicable to circular column only.

2.6 SEISMIC RETROFIT GUIDELINES IN CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC-2010)

Section 4.11 of CHBDC (2010) provides requirements for seismic evaluation of existing bridges. Section 4.11 specifies that, for lifeline bridges special studies shall be performed to evaluate their seismic performance. The earthquake level and procedure used for evaluating lifeline bridges shall be specified by the Owner and shall comply with the requirements described in CHBDC (2010). For emergency-route and other bridges, the requirements of Clause 4.11 shall apply. CHBDC (2010) requires that seismic rehabilitations shall be designed so that a minimum level of safety is provided. This level shall be (a) comparable to that intended for new bridges; or (b) as prescribed by the Regulatory Authority.

CHBDC recommends that for each structural element and connector, the required response modification factor, R_{req} , should be determined. The required response modification factor, R_{req} shall be determined using the following equation.

$$R_{req} = S_e/C \qquad [2.1]$$

Where,

S_e = seismic force effect assuming all members remain elastic, calculated in accordance with Clause 4.11.5, except as limited by capacities of other members

C = member reserve capacity after the effects of dead load have been considered, calculated in accordance with Clause 4.11.8.

The response modification factor, R, for the rehabilitated ductile substructure element shall be determined accordingly but shall not exceed the smaller of (a) the value of R from Table 4.5 (section 4.4.8) corresponding to the type of substructure element; or (b) 5.0.

CHBDC-2010 focuses on some major aspects that should be considered in designing the retrofitting of existing bridges which are as follows:

- The increased stiffness due to retrofitting may attract larger seismic forces in some elements.
- Retrofitting measures should be designed to prevent damage of inaccessible regions such as underground foundations.
- The durability of retrofit measure should be assessed.
- Retrofitting applied in sequence must have proper planning of the sequences.
- Strengthening of some members may result in larger force demand on other members, connection, and foundation or superstructure elements.

In the design of the rehabilitation measures, CHBDC (2010) requires the following measures:

- column rehabilitation jackets shall terminate 100 mm from the top of the footing and the bottom of the cap beam;
- if uplift occurs near the base of a structure, care shall be taken to this movement. Due consideration shall be given to other effects, e.g., loss of support and impact;
- if base isolators are employed, care shall be taken in assessing the structural stability at other limit state combinations (e.g., wind);
- the durability of the rehabilitation measures shall be assessed; and
- a complete re-analysis of the rehabilitated structure in both the longitudinal and transverse directions shall be conducted to assess the performance of the rehabilitated structure.

CHAPTER 3 : DEVELOPMENT OF A PRIORITIZATION METHODOLOGY FOR MAINTAINING BRIDGE INFRASTRUCTURE SYSTEMS

3.1 GENERAL

In a transportation system highway bridges are the essential nodes joining highways. But these critical elements undergo severe condition and functional deterioration due to aggressive environmental conditions, increasing traffic volumes and truck loads. Recent collapse of a number of bridges in North America indicates their increased risk of failure. Keeping these deteriorated and aged bridges within the acceptable safety and serviceability limit is a major concern for the highway agencies. These deteriorated bridges are the most vulnerable components of highway networks and require regular maintenance and rehabilitation work. Failure to perform regular maintenance work may result in sudden collapse of a bridge, which can severely affect the traffic in the road network and even cause severe damages to human lives. In addition, this might draw the traffic to follow an alternate route thus incurring extra miles to drive. Therefore, it is imperative to keep the bridges safe and operational. But these maintenance and rehabilitation works require large financial and human resources which are not always available in required amount to the transportation agencies. In Canada, municipal transportation and transit infrastructure comprised 55% of total municipal infrastructure and municipalities need around $21 billion to maintain and upgrade existing transportation infrastructure assets (Mirza, 2007). America’s surface transportation systems had a deficiency of nearly $ 130 billion estimated in 2010. This deficiency has and will have serious impact on the overall US economy.

If the present trend continues, the current deficiency will increase by 82% to $210 billion by 2020, which will leave a significant and mounting burden on overall economy (ASCE, 2011).Under such conditions it is very difficult to upgrade severely deteriorated infrastructure to an acceptable level. To maintain an optimum balance between available resources and desired performance it is required to use these limited resources in a timely and effective way. In order to maintain a sustainable bridge network, it requires a complex decision making process to prioritize bridges for maintenance works.

Common causes of deficiencies that are gradually deteriorating the existing bridges are generally agreed among field researchers and practitioners as fatigue, corrosion, settlement, scouring, etc. (Feld and Carper 1997, Nakatani et al. 2004). Although resources are allocated each year for repair, rehabilitation and/or replacement of bridges, the budget is limited and covers about 30% to 70% of the actual need. Identifying particular bridges for such operations is a daunting task as it involves numerous inter-related decision parameters which end up causing dilemmas to the owner and its agencies. Very often bridge engineers and policy makers are being continuously pressed to justify the funding order proposed for rehabilitation and maintenance work. Such situation often invites an unacceptable level of structural and functional performance degradation resulting in bridge failure. Until the deteriorated and aged bridges are rehabilitated, levels of bridge safety and physical conditions would be significantly lowered over time.

For the last few decades bridge engineers and researchers have developed tools for bridge condition assessment and plans for maintenance budget allocation. A number of researchers developed bridge condition assessment tools using deterministic approach (Frangopol & Hearn, 1996), probabilistic approach (Thoft-Christensen, 1995; Val & Melchers, 1997; Dabous et al. 2008; Sun and Sun, 2011) and fuzzy logic based approaches (Kawamura and Miyamoto, 2003;

Sasmal et al. 2006). Lounis (2006) developed a multi-objective optimality index to prioritize deficient bridge decks for maintenance. Sasmal and Ramanjaneyulu (2008) developed a decision making tool for priority ranking of bridges. They developed fuzzy logic based AHP approach for condition assessment of deteriorating bridges. The prioritization method proposed in this study develops a knowledge based decision support tool, which has a major interest for government in the federal and provincial levels, consultants and also municipality level decision makers. This method emphasizes the importance of bridge infrastructure to the overall economy, performance level of bridges, social importance of bridges, etc., for simulating maintenance priority orders. This decision support tool utilizes a knowledge based evaluation technique by considering location and topology of the bridge, contribution to GDP, traffic volume, alternate route, and bridge condition and to develop a Bridge Prioritization Index (*BPI*). The *BPI* will aid decision makers in the allocation of emergency funds based on detailed information of the bridge.

The goal of this particular study is to develop and exercise a methodology for prioritizing bridge structures and develop a *BPI*. This will aid in selecting maintenance strategies and accommodating a limited budget for a network of bridges. To accomplish these objectives a decision support tool is developed that requires minimum amount of data, easy to use and suitable for network level project application. In order to provide a robust prioritization methodology three relevant indices i.e. importance factor index, cost index and bridge condition index are selected to identify the areas of prime concern when selecting bridges for rehabilitation and maintenance. Figure 3.1 shows the generic framework proposed in this study for developing *BPI*. Within the methodology, a variety of techniques and multi-criteria decision making tools are utilized to accomplish the stated objectives, which are as follows:

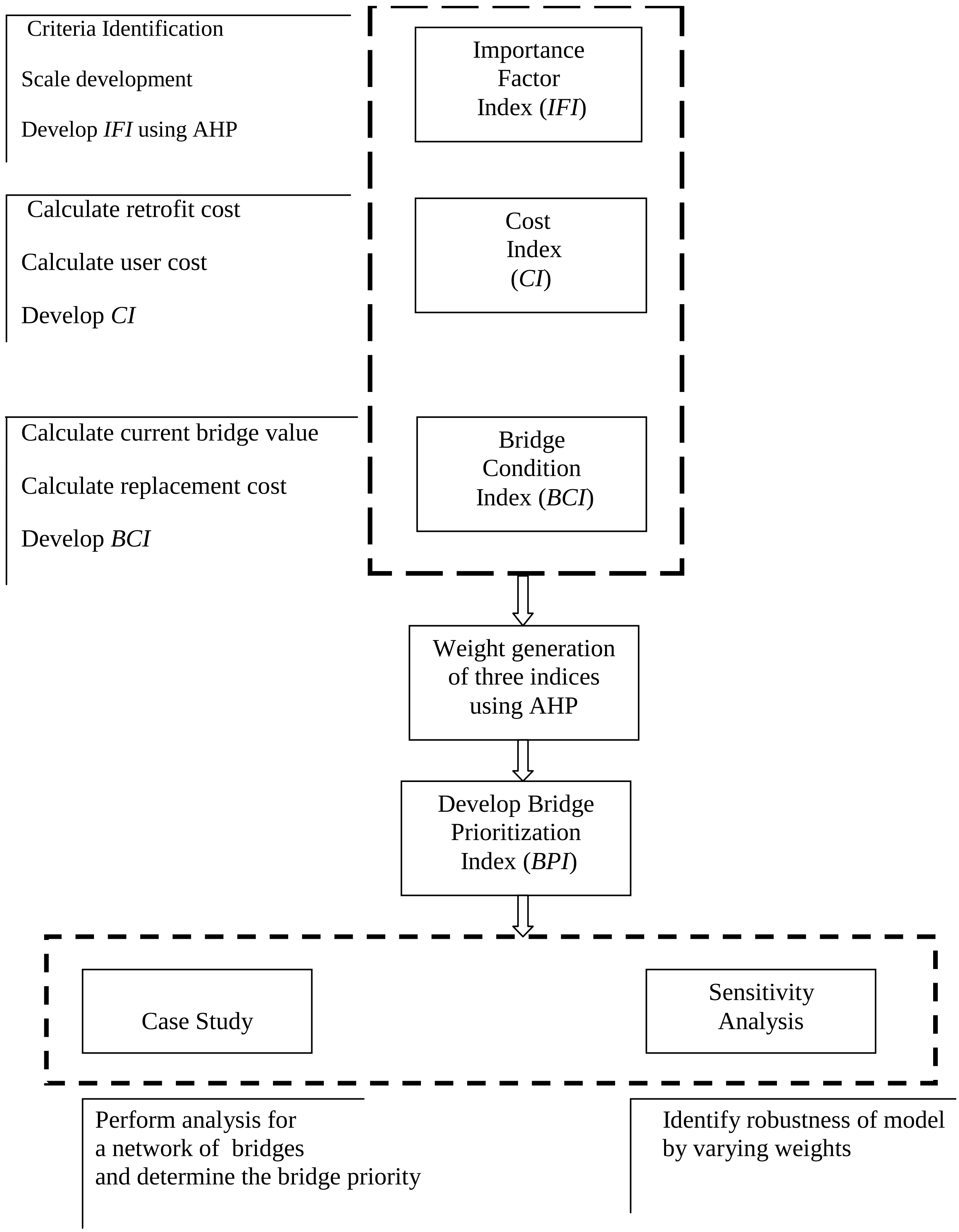


Figure 3.1: Proposed generic framework for development of bridge prioritization index (*BPI*)

- Review of the-state-of-the-art and current methods used to prioritize bridges and distribute maintenance funds.

- Establish important criteria to develop the prioritization index.

- Formulation of three indices and development of semantic scale for each variable based on available literature and expert consultation.

- Calibration of bridge prioritization index based on the established criteria and using bridge inspection data.

- Demonstration of developed methodology on a road network of ten bridges in British Columbia, Canada using actual data as a case study.

3.2 REVIEW OF CURRENT PRIORITIZATION METHOD

Project prioritization and fund allocation is a cumbersome and tedious work for highway agencies. An effective and efficient decision making tool for optimizing maintenance prioritization can aid decision makers to overcome these problems. To date various prioritization procedures have been developed by researchers and highway agencies to prioritize maintenance and rehabilitation projects. A literature review is carried out along with a detailed examination of the seismic prioritization procedures adopted in the United States of America, Canada, New Zealand and other procedures suggested by numerous researchers. In the following sections each of these methodologies is briefly reviewed.

The prioritization methods developed by different highway agencies and researchers are summarized in Table 3.1. A comparative analysis of the methods reviewed is also presented in Table 3.2.

Table 3.1: Review of existing prioritization methods

Methods	Factors Considered	Decision Criteria
CALTRANS, 1992	Load Factor: Magnitude, acceleration, duration, soil condition at site. Structural Factor: Number of hinges, year of construction, Number of columns per bent, Outriggers. Social Factor: Average daily traffic, Route type, Lifeline, Miles to detour	A global utility function is derived for each attribute and they are multiplied with weight of each individual attribute to determine the final ranking. The weights are based on expert opinion survey.
ATC-6-2, 1983	Three major factors: Vulnerability of the structural system, Seismicity of the bridge site, Importance of the bridge.	Two tier process involving preliminary screening followed by quantitative evaluation. A Seismic rating system ranging from 0 to 10 is used for ranking.
Washington DOT (Babei & Hawkins, 1991)	Criticality Factor: Route type, ADT, Utility, Detour length, Bridge length. Vulnerability factor: Bedrock acceleration, Remaining life and structural vulnerability	Ranking of the bridge is derived from the multiplication of criticality factor and vulnerability factor. The priority index ranges between 1 to100.
Nevada DOT (Sanders et al., 1993)	Importance factor: traffic, detour length, route type, utility lines, defence route and rail road. Vulnerability Factor**:** Structural vulnerability, bed rock acceleration and soil effects.	Importance factor ranges from 1 to 17, while vulnerability ranges from 1 to 14 and over all risk score derived from this two varies from 10 to 1000.

(Continued)

Methods	Factors Considered	Decision Criteria
Transport Quebec (1996)	Function index, condition index and vulnerability index.	Priority is determined using a multi attribute decision theory.
Illinois DOT (Cooling et al. 1992)	Seismic risk Factor: probability of failure and consequences of failure. Consequence of failure is derived using multi-attribute value function.	Risk based two stage approach. Bridges are ranked in descending order of the priority score.
FHWA (1995)	Vulnerability factors, Seismic hazard factors and Socio-economic factors.	Priority index is calculated as a function of bridge rank, importance, social and economic factors.
Saskatchewan Ministry of Highways and Infrastructure (SMHI, 2009)	Condition, traffic volume, load capacity, road way function, rehabilitation category and special consideration.	Two tire evaluation process. First stage determines the eligibility of the selected projects and second stage prioritizes the eligible projects.
Valenzuela et al., 2010	Seismic risk, strategic importance, bridge condition and hydraulic vulnerability.	Priority is determined using a mathematical formula consisting of several factors and expert consultation.
Dabous and Alkass, 2010	Condition, load capacity, ADT, Seismic risk, road type, Vertical clearance, Approach condition, Drainage system.	Ranking method developed using multi-attribute utility theory. Bridges are ranked in terms of their utility value.
Basoz and Kiremidjian, 1995	Vulnerability and Importance.	A value model is developed to determine the priority which integrates vulnerability and importance criteria.

Table 3.2: Comparative analysis of existing prioritization methods

Methods	Disadvantages	Advantages
CALTRANS, 1992	Does not consider the structural condition and economic impact into account.	Does not require massive data supported by statistical distribution and applicable for quick decision making process.
ATC-6-2, 1983	Considers only the technical aspects and does not include economic, administrative and political factors.	Bridges are classified according to seismic performance category considering the structural importance, acceleration coefficient and type of bridge.
Washington DOT (Babei & Hawkins, 1991)	Criticality and vulnerability have the same weight in the priority model. Does not follow any procedure for weight assignment.	Bridges are divided into five groups based on their structural deficiencies. Then bridges of each group are ranked in terms of their importance criteria.
Nevada DOT (Sanders et al.,1993)	No comparison of retrofit and replacement cost is included which is necessary for high ranking bridges.	It incorporates both the route and ADT under the structure which is neglected by many DOT.
Illinois DOT (Cooling et al. 1992)	No consideration for traffic volume, economic factors and factors related to retrofit cost. Ranking obtained by multiplication of two main component may distort the final ranking.	Two stage approach allows more detailed evaluation and investigation of features that performed poorly in past earthquakes.

(Continued)

Methods	Disadvantages	Advantages
FHWA (1995)	Does not consider the condition of bridge elements.	Bridges are classified into four groups according to acceleration coefficient and importance category, which helps to identify bridges requiring retrofit.
Transport Quebec (1996)	Ignores the economic factor, benefit-cost ratio, soil condition and element vulnerability.	Takes into account bridge and route type, traffic volume, detour length, foundation condition and structural behaviour.
Saskatchewan Ministry of Highways and Infrastructure (SMHI, 2009)	Priority ranking is determined by summing up the relative importance of each evaluation factor. This simple addition process may raise difficulty for a large group of bridges.	Two stage evaluation process helps to effectively manage projects and prioritize actual potential bridges for rehabilitation.
Valenzuela et al., 2010	Requires a complex estimation of seismic risk from historical seismic data and seismic zoning.	This method is applicable for network of bridges.
Dabous and Alkass, 2010	Does not consider economic and social factors.	Applicable to any network of bridges, flexible in adjusting the weights and modifying the utility functions.
Basoz and Kiremidjian, 1995	Requires extensive calculation of probability and network analysis. Does not consider the cost components.	Applicable for network bridges which is considered in the importance factor.

3.3 OUTLINE OF THE PROPOSED PRIORITIZATION METHOD

This study proposes the *BPI* to decide prioritization of preventive maintenance work for certain bridges in a specific bridge network. Figure 3.2 shows the simple hierarchical structure for developing *BPI*. This structure demonstrates the formation of *BPI* resulting from the aggregation of the Importance Factor Index (*IFI*), Cost index (*CI*) and Bridge Condition Index (*BCI*).Each index is divided into some criteria and sub-criteria (Figure3.2). The hierarchical structure (Figure 3.2) consists of three modules. Each module corresponds to development of three indices, *IFI*, *BCI* and *CI*. Once these three indices are developed, they are multiplied by their weighting coefficients to obtain the *BPI* using equation (3.1a).

$$BPI = \alpha_{IFI} IFI + \alpha_{CI} CI + \alpha_{BCI} BCI \qquad [3.1a]$$

$$\text{where, } \alpha_{IFI} + \alpha_{BCI} + \alpha_{CI} = 1 \qquad [3.1b]$$

A simple linear equation is proposed here for determining *BPI*. Similar linear equation has also been used by several transportation departments e.g. (SMHI, 2009; Transit New Zealand, 1998). Although the proposed equation is very simple it includes all the parameters that can affect the bridge ranking. The *IFI* and *BCI* are determined based on the individual bridge characteristics and the condition state of each bridge. Meanwhile, the *CI* is determined from the total cost required for rehabilitation of the bridge and the user cost incurred during the rehabilitation period.

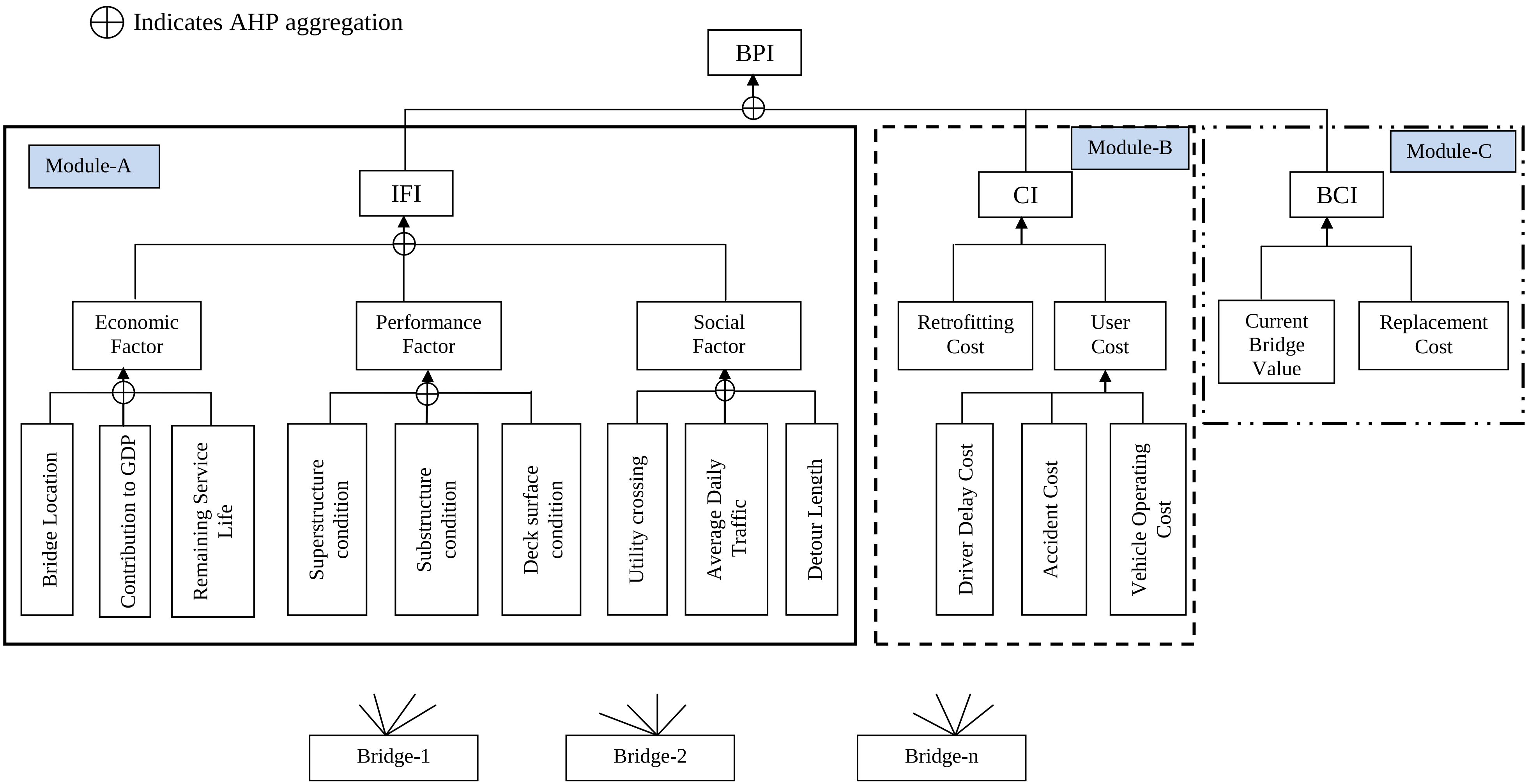


Figure 3.2: Hierarchical framework for development of *BPI*

The prioritization weighting coefficients of the *IFI* (α_{IFI}), the *BCI* (α_{BCI}) and the *CI* (α_{CI}) (with their total) are determined by reflecting bridge location, traffic condition, social preference, etc. They are decided specifically by local practitioners. In this study, the weighting coefficients are determined using Analytical Hierarchy Process (AHP) developed by Saaty (1980). In the course of bridge maintenance planning, the bridges with higher *BPI*s are given higher priority so that the preventive maintenance work on the bridges with higher *BPI*s is executed in the early years within the budget limitation and the work on the bridges with lower *BPI*s is postponed.

3.3.1 Importance Factor Index (*IFI*)

The importance factor determines the importance of the bridge in the overall road network. At first, the *IFI* is obtained from three main-indicators, the Economic Factor (*EF*), Performance Factor (*PF*) and Social factor (*SF*). AHP is employed here to determine the *IFI*. AHP is based on an axiomatic foundation and has diversified application due to its simplicity and ability to deal with complex decision making problems. Module-A in Figure 3.2 represents the hierarchy for developing *IFI*. Major criteria of Module-A are broken down to their basic attribute level (Figure 3.2).

3.3.1.1 Economic Factor (*EF*)

Economic Factor (*EF)* represents the special characteristics of the individual bridge in the road network. The index *EF* is obtained from three sub factors, bridge location (*BL*), contribution to GDP (*CG*) and Remaining Service Life (*RSL*). The *BL* value is determined by the location of the bridge, and the location is divided into three areas of dense, normal, and remote. The *BL* index is estimated using Table 3.3. The index *CG* is determined on the basis of contribution to the GDP and is divided into three groups namely, large, medium and small and this index is determined according to Table 3.3. This *CG* value is based on how the bridge is contributing to GDP based on its proximity to industrial growth or any place of economic development. The index *RSL* is obtained from the remaining service life of the bridge assuming a life cycle of 75 years. The bridges are divided into three groups as per their *RSL* as shown in Table 3.3. All the scores developed here are subjective in nature and were derived by taking expert opinion and from existing methods in practice by various highway agencies.

Table 3.3: Bridge score based on location, *RSL* and *CG*

Bridge location	**Contribution to GDP**	**Remaining service life**	**Score**
Dense Area	Large	0-25	5
Normal Area	Medium	25-50	3
Remote Area	Small	>50	1

3.3.1.2 Performance Factor (*PF*)

Performance Factor *(PF)* represents the physical and serviceable conditions of a bridge and is an important indicator in the decision of maintenance prioritization. While healthy bridges need less attention, deteriorated bridges need more attention to ensure their safety and functionality. Condition inspection data is viewed as the most superior because it captures the severity and extent of the condition of bridge elements. In this study the condition values are used to evaluate the *PF*, which in turn determine the project eligibility and influence the allocation of funding. The *PF* index is obtained from three sub-indicators, the substructure condition (*SC*), the condition of superstructure (*CS*) and the deck surface (*DS*) condition. Each sub-indicator is further classified as *excellent, good, fair, poor* and *very poor* according to the existing condition determined by visual inspection and condition assessment. According to the condition they are assigned score from 1 to 5 as shown in Table 3.4.

Table 3.4: Bridge score based on condition, *ADT* and detour length

Condition	ADT	Detour length (km)	Score
Excellent	500-1000	0-10	1
Good	1000-5000	10-20	2
Fair	5000-10000	20-30	3
Poor	10000-20000	30-50	4
Very poor	>20000	>50	5

3.3.1.3 Social Factor (*SF*)

Social Factors *(SF*s) which are often outside the realm of engineering may have the ultimate influence on the decision to prioritize a bridge. When the preventive maintenance work is executed on certain bridges, the bridge authorities and traffic users regard this work as being able to generate benefit for whole road segments and the society as well. The *SF* index is obtained from three sub factors: Traffic volume (*TV*), Detour Length (*DL*) and Utility crossing (*CU*) over the bridge. Here, Average Daily Traffic (*ADT*) is used to measure the traffic volume (*TV*) and categorized in five different types as shown in Table 3.4. The *DL* is expected to play a major role in bridge prioritization. The factor *DL* dictates how much extra miles to be travelled if the bridge under consideration is closed. It is classified in five different classes based on the length of detour (Table 3.4). The factor *CU* is determined whether there is any utility service line passing through the bridge such as gas or water transmission line, telephone and electrical services etc. Based on this information the bridges are assigned with a score as per Table 3.5.

Table 3.5: Bridge score based on utility crossing

Utility crossing	**score**
Yes	3
No	1

3.3.2 Cost Index (*CI*)

Budget constraints and escalating costs of maintenance actions require extra attention to maintain bridge structures functioning at an acceptable level. Project cost analysis provides the means to address this issue. Cost analysis is an essential engineering economic analysis tool to compare competing alternatives for projects with entailing costs and benefits that stretch over long spans of time. It does not address equity issues rather incorporates all of the costs agency and user incurred during the service life of alternative investments. A cost analysis may be conducted not only at the project development stage of a new construction but also in selecting maintenance strategies and competing alternatives in a network of bridges. This study develops the Cost Index (*CI)* based on the cost analysis of the competing bridges. In this study, the cost associated with bridge rehabilitation and the user cost incurred during the rehabilitation period is considered. The total cost is assumed to be the summation of these two costs. Considering the predicted schedule of maintenance activities and their associated agency and user costs the projected cost for each design alternative is determined (FHWA, 2004). Cost analysis thus can impact the decision making process by providing critical information. Therefore, this cost analysis possesses enormous potential for the highway agencies to estimate the total expected costs for a group of bridges and minimize the total maintenance costs.

3.3.2.1 Cost of Retrofitting

Retrofit cost is essentially important for the development of the proposed prioritization technique. However, retrofit cost is meant to be an empirical formula that is a complicated function of several factors. The most important factors are the number of expansion joints, the height of columns, water crossing, soil profile types, the duration and intensity of maximum credible earthquake. The study of retrofit costs is very complex and needs careful estimation. In this study the retrofit cost is calculated with the help of an expert estimator. Looking at the bridge inspection inventory it was decided which elements of a particular bridge need to be retrofitted or rehabilitated. Then, the unit price of each element retrofit was obtained from a construction agency. Based on the available data and unit price for each element retrofitting, the total retrofit cost for each bridge was determined.

3.3.2.2 User Cost (*UC*)

User costs (*UCs*) are encountered by the user of the project. *UC*s are highly influenced by the roadway traffic characteristics. For instance, bridge rehabilitation or maintenance program often causes congestion and delays for private and commercial traffic. Maintenance and repair of an existing bridge often result in capacity restrictions, as well as extra mileage caused by detours. When the traffic flow in the maintenance area increases the bridge operates under force-flow condition. This can impact drivers' personal time as well as the operating cost of vehicles sitting in traffic. Very often accidents take place in areas nearby the work zone involving harm to both vehicles and human lives.

These traffic delay cost, vehicle operating cost and accident cost can be computed using simple formulas (Ehlen, 1997). Three types of user costs are usually computed:

Driver delay cost (*DDC*): the personal cost to driver delayed by road work.

Vehicle operating cost (*VOC*): The capital costs of vehicle delayed by road work.

Accident costs (*AC*): the cost of damage to vehicles and human due to the road work.

The total user costs can be expressed as:

$$UC = DDC + VOC + AC \qquad [3.2a]$$

Here, *VOC* includes the costs of fuel, tyres, engine oil, maintenance, and depreciation, *AC* includes costs related to fatal accidents, non-fatal injury accidents, property damage accidents, and *DDC* is considered as a function of the hourly wage rate. In most cases *DDC* is the most relevant component.

The driver delay cost can be calculated as:

$$DDC = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) ADT \,.\, N \,.\, w \qquad [3.2b]$$

Where, L= length of the affected roadway, S_a= traffic speed during bridge work activity, S_n= normal traffic speed, ADT= average daily traffic, N= No of days of roadwork and w= hourly time value of drivers.

The vehicle operating cost can be calculated as:

$$VOC = \left(\frac{L}{S_a} - \frac{L}{S_n}\right) ADT \,.\, N \,.\, r \qquad [3.2c]$$

r = weighted average vehicle cost

The accident cost can be calculated as:

$$AC = L \cdot ADT \cdot N \cdot (A_a - A_n) \cdot c_a \qquad [3.2d]$$

Where, c_a= cost per accident, A_a and A_n are the during construction and normal accident rate per vehicle kilometre.

3.3.3 Bridge Condition Index (*BCI*)

The Bridge Condition Index (*BCI)* is a measure of the average condition of a bridge and its components. California Bridge Health Index (*BHI*) developed by Johnson and Shepard (1999) is a practical example of a bridge condition index. This method develops the *BHI* considering the element inspection inventory and calculates the remaining asset value for an individual or a network of bridges. This is done by determining the remaining asset value of a bridge and relating with current element condition states and element failure costs. In Canada, *BCI* was developed by the Ministry of Transportation, Ontario for the Ontario Bridge Management System (OBMS). This *BCI* largely depends on the information derived from regular inspections of the bridge and its components. Functionality and performance of the bridge is affected by the condition and need to be addressed during rehabilitation program. *BCI* helps to develop a strong link between the bridge condition and investment decision. *BCI* is an effective index as it relates the overall condition of all bridge elements together while considering the remaining economic value of the bridge as well. The *BCI* value ranges from 0 to 100. Bridge with a *BCI* over 70 is considered as in good condition and does not require any maintenance work within the next five years. Bridges with *BCI* value between 60 and 70 is considered as in fair condition. Bridges in fair condition will require a major maintenance within next few years. If the *BCI* value is less than 60, bridges are in poor condition and they require immediate attention. Dividing the current value of the bridge by the bridge replacement cost, the *BCI* is calculated. The replacement cost is determined based on the cost of new bridge construction. The *BCI* value is calculated as follows:

$$BCI = (\textit{Current value}/\ \textit{Replacement cost})\ X\ 100 \qquad [3.3]$$

Bridges not currently in good or excellent condition are those needing rehabilitation. In this study the *BCI* values were determined using equation (3.3). *BCI* is a good indicator of urgency of repair work required.

3.4 CASE STUDY

The British Columbia Ministry of Transportation and Infrastructure is responsible for 400 km of provincial Disaster Response Routes and 900 highway structures in the highest seismic zone of Canada. An earthquake of moderate intensity can result in extensive damage and a potential collapse could result from a major earthquake. The loss of any portion of these routes could have significant impact on emergency response efforts, and negatively affect public well being. The effects would be felt across the nation and for many years into the future. There are around 3000 bridges in BC. Many of them were constructed before 1965 and they are on the verge of their full service life. Existing bridges are generally in poor condition with many defects such as corrosion, fatigue, ageing, thermal gradient, freeze-thaw cycle and other environmental loads, human invasion, construction defects, and scouring. Bridge rehabilitation program is currently executed randomly based on the condition inspection and equal-consideration to the candidate bridges. The provided budget is enough to execute only 30% to 50% of the actual demands. Therefore, site maintenance is conducted only on a limited number of bridges. The objective of this study is to propose an index for a bridge prioritization in the context of a need-based framework. The *BPI* is developed as an aid to prioritize bridges for fund allocation. In order to apply the methodology discussed earlier, ten reinforced concrete bridges of the British Columbia Ministry of Transportation and Infrastructure network were analyzed. The bridge inventory and detailed inspection report was obtained from BC MoT. Due to confidentiality the details of the bridges are not provided in this thesis. The following sections demonstrate the development of *BPI* for the ten bridges adopted in this study.

3.4.1 Development of Importance Factor Index

The *IFI* determination was decomposed into the four-level hierarchy shown in Module-A of Figure 3.2. The highest level is the main objective that is to identify the *IFI* of the bridges considered. The elements of the second level are the 3 multiple criteria identified for the case study. Each element of the second level is decomposed into 3 sub criteria. The elements of the lowest level are the 10 candidate bridges selected from the bridge inventory provided by BC MoT. Considering the factors shown in Module-A (Figure 3.2) and the values provided in Tables 3.3-3.5 the decision matrix (Table 3.6) is obtained.

A quantitative evaluation of the relative importance (weight) of each criterion to the final decision is needed. The weights will amplify or de-amplify the evaluations of the alternatives in order to reflect how much each criterion is important relatively to the others in the choice of the best solution. The decision matrix of Table 3.6 is normalized using equation (3.4) to get the normalized decisions matrix (Table 3.7)

$$P_{ij} = \frac{y_{ij}}{\sum_{i=1}^{n} y_{ij}} \qquad [3.4]$$

Table 3.6: Decision matrix

Alternative	*BL*	*CG*	*RSL*	*CS*	*SC*	*DC*	*CU*	*ADT*	*DL*
B1	3	1	5	2	3	2	1	3	2
B2	5	5	5	1	1	2	1	3	4
B3	5	5	5	1	1	4	1	3	3
B4	3	3	5	3	1	2	1	3	5
B5	5	5	3	3	2	1	1	4	1
B6	1	3	3	1	1	3	1	2	1
B7	3	3	1	1	3	1	1	3	1
B8	5	3	1	1	1	4	3	4	1
B9	5	5	1	1	2	1	1	3	3
B10	5	3	1	1	1	2	1	4	1

Table 3.7: Priority vectors for bridges with respect to each criterion

Alternative	*BL*	*CG*	*RSL*	*CS*	*SC*	*DC*	*CU*	*ADT*	*DL*
B1	0.075	0.028	0.167	0.133	0.188	0.091	0.083	0.094	0.091
B2	0.125	0.139	0.167	0.067	0.063	0.091	0.083	0.094	0.182
B3	0.125	0.139	0.167	0.067	0.063	0.182	0.083	0.094	0.136
B4	0.075	0.083	0.167	0.200	0.063	0.091	0.083	0.094	0.227
B5	0.125	0.139	0.100	0.200	0.125	0.045	0.083	0.125	0.045
B6	0.025	0.083	0.100	0.067	0.063	0.136	0.083	0.063	0.045
B7	0.075	0.083	0.033	0.067	0.188	0.045	0.083	0.094	0.045
B8	0.125	0.083	0.033	0.067	0.063	0.182	0.250	0.125	0.045
B9	0.125	0.139	0.033	0.067	0.125	0.045	0.083	0.094	0.136
B10	0.125	0.083	0.033	0.067	0.063	0.091	0.083	0.125	0.045

In order to find out the priority vectors associated with each sub-factors pair wise comparison of sub-factors for the main factor are carried out. For generating weights, pair-wise comparison of different attributes and criteria are carried out following the scale of Saaty(1980) as presented in Table 3.8.

The procedure is as follows:

- Developing pair-wise comparison matrix. (Table 3.9)
- Synthesizing the pair-wise comparison matrix (example: Table 3.10);
- Calculating the priority vector for a criterion such as Economic factor (Table 3.10);
- Calculating the consistency ratio;
- Calculating the eigen value (λ_{max});
- Calculating the consistency index, (*IC*);
- Selecting appropriate value of the random consistency ratio from Table 3.11; and
- Checking the consistency of the pair-wise comparison matrix to check whether the decision maker's comparisons were consistent or not.

Synthesizing the pair-wise comparison matrix is performed by dividing each element of the matrix by its column total. The priority vector in Table 3.12 can be obtained by finding the row average. For calculating the consistency ratio (*CR*), the eigen value (λ_{max}) of the matrix was calculated. The Consistency Index (*IC*) was obtained using equation (3.5)

$$IC = \frac{\lambda max - n}{n-1} \qquad [3.5]$$

Where, *n*= size of the matrix.

Table 3.8: Pair-wise comparison scale for AHP preferences (Saaty, 1980)

Numerical value	Verbal judgement of preferences	Explanation
9	Extreme importance	The evidence favouring one activity over another is of the highest possible order of affirmation
8	Very, very strong	
7	Very strong or demonstrated	An activity is favoured very strongly over another; its dominance demonstrated in practice
6	Strong plus	
5	Strong importance	Experience and judgment strongly favour one activity over another
4	Moderate plus	
3	Moderate importance	Experience and judgment slightly favour one activity over another
2	Weak or slight	
1	Equal importance	Two activities contribute equally to the objective

Table 3.9: Pair-wise comparison matrix for economic factor

Economic factor	***BL***	***CG***	***RSL***
BL	1.000	0.250	0.143
CG	4.000	1.000	0.250
RSL	7.000	4.000	1.000

λmax= 3.067; *IC*= 0.0335; *CR*=0.057

Appropriate value of random index, *RI*, for a given matrix size is determined using Table 3.11. Then the Consistency Ratio (*CR*) is calculated as:

$CR = \frac{IC}{RI}$ [3.6]

Similarly, the pair-wise comparison matrices and priority vectors for the performance factor and social factor can be found as shown in Tables 3.12 and 3.13, respectively. As the value of *CR* is less than 0.1, in all three cases the judgments are acceptable.

Table 3.14 includes the *IFI* of the bridges. The *IFI*s were obtained by multiplying each column in the bridge local priority matrix (Table 3.7) by the priority of the corresponding criterion (Table 3.10, 3.12 and 3.13) and adding across the rows.

Table 3.10: Synthesized matrix for economic factor

Economic factor	***BL***	***CG***	***GC***	**Priority vector**
BL	0.083	0.048	0.103	0.078
CG	0.333	0.190	0.179	0.234
GC	0.583	0.762	0.718	0.688

Table 3.11: Random index (*RI*) (Saaty, 1980)

Size of Matrix	1	2	3	4	5	6	7	8	9	10
RI	0	0	0.58	0.9	1.12	1.24	1.32	1.41	1.45	1.49

Table 3.12: Pair-wise comparison matrix for performance factor

Performance factor	***CS***	***SC***	***DC***	**Priority vector**
CS	1	2	6	0.587
SC	0.5	1	4	0.324
DC	0.167	0.25	1	0.089

λ_{max}= 3.009; *IC*= 0.0045; *CR*=0.007

Table 3.13: Pair-wise comparison matrix for social factor

Social factor	*UC*	*ADT*	*DL*	Priority vector
UC	1	0.143	0.25	0.077
ADT	7	1	5	0.709
DL	4	0.2	1	0.214

λ_{max}= 3.081; *IC*= 0.0405; *CR*=0.069

Table 3.14: Importance factor index (*IFI*)

Criteria	*BL*	*CG*	*RSL*	*CS*	*SC*	*DC*	*CU*	*ADT*	*DL*	*IFI*
					Criteria priorities					
	0.078	0.234	0.688	0.587	0.324	0.089	0.077	0.709	0.214	
Alternative						**Alternative priorities**				
B1	0.075	0.028	0.167	0.133	0.188	0.091	0.083	0.094	0.091	0.366
B2	0.125	0.139	0.167	0.067	0.063	0.091	0.083	0.094	0.182	0.336
B3	0.125	0.139	0.167	0.067	0.063	0.182	0.083	0.094	0.136	0.335
B4	0.075	0.083	0.167	0.200	0.063	0.091	0.083	0.094	0.227	0.407
B5	0.125	0.139	0.100	0.200	0.125	0.045	0.083	0.125	0.045	0.378
B6	0.025	0.083	0.100	0.067	0.063	0.136	0.083	0.063	0.045	0.222
B7	0.075	0.083	0.033	0.067	0.188	0.045	0.083	0.094	0.045	0.235
B8	0.125	0.083	0.033	0.067	0.063	0.182	0.250	0.125	0.045	0.245
B9	0.125	0.139	0.033	0.067	0.125	0.045	0.083	0.094	0.136	0.251
B10	0.125	0.083	0.033	0.067	0.063	0.091	0.083	0.125	0.045	0.224

3.4.2 Development of Cost Index

The *CI* was determined following the procedure discussed in section 4.2. Table 3.15 shows the cost of retrofitting for the ten bridges considered in this study. The user cost associated with each bridge rehabilitation program is shown in Table 3.16. This table also shows the total cost for each bridge rehabilitation project.

3.4.3 Development of Bridge Condition Index

The *BCI* values for the selected ten bridges were calculated following the procedure described in section 3.3.3. Table 3.17 shows the *BCI* for the 10 bridges considered in this study. From Table 3.17 it is evident that all the bridges have *BCI* value below 60. It indicates that all the bridges are in poor condition and require immediate rehabilitation.

Table 3.15: Retrofitting cost

Bridge No	**Retrofitting cost ($)**
B1	482160
B2	1790560
B3	871840
B4	223840
B5	376480
B6	1648000
B7	1624960
B8	2618560
B9	2471680
B10	704800

Table 3.16: Total retrofitting cost

Bridge No	Length (km)[1]	*Sn*[1] (km/h)	*Sa*[1] (km/h)	*ADT*[1]	*N*	*W*[2]	r[3]	*Ca*[4]	*DDC* ($)	*VOC* ($)	*AC* ($)	Total Cost ($)
B1	0.165	80	35	8000	21	15	8	154000	6682.5	3564	4695	497102
B2	0.0079	70	30	8000	21	15	8	154000	379.2	202.24	224	1791366
B3	0.0178	80	35	8000	21	15	8	154000	720.9	384.48	506	873452
B4	0.0092	80	35	8000	21	15	8	154000	372.6	198.72	261	224673
B5	0.0415	75	30	15000	21	15	8	154000	3921.75	2091.6	2214	384708
B6	0.018	80	35	5000	21	15	8	154000	455.625	243	320	1649019
B7	0.019	50	25	8000	21	15	8	154000	957.6	510.72	540	1626969
B8	0.064	60	25	15000	21	15	8	154000	7056	3763.2	3415	2632794
B9	0.097	110	50	8000	21	15	8	154000	2666.618	1422.19	2760	2478529
B10	0.0381	90	40	15000	21	15	8	154000	2500.313	1333.5	2033	710667

Source:1)BC MoT; (2)Pay scale Canada; (3) Driving cost and (4)Transport Canada. A_n= 7.4 per million per vehicle km, A_a= 8.5 per million per vehicle km (Source: Transport Canada.

Table 3.17: Bridge condition index

Bridge No	*BCI*
B1	42.8
B2	27.4
B3	35
B4	45.4
B5	40.8
B6	32.6
B7	34
B8	25
B9	26.4
B10	35.6

3.4.4 The Bridge Prioritization Index (*BPI*)

After obtaining the *IFI*, *CI* and *BCI* the bridge with worst condition among the existing bridges considered in the study can be easily identified. The *BPI* is obtained by weighted sum method. The prioritization weighting coefficients of the importance factor index (α_{IFI}), the bridge condition index (α_{BCI}) and the cost index (α_{CI}) with their total ($\alpha_{IFI} + \alpha_{BCI} + \alpha_{CI=} 1$) are determined by another AHP. Table 3.18 shows the pair-wise comparison matrix of the three main criteria i.e. *IFI*, *CI* and *BCI*. The weighting coefficients obtained are: α_{IFI} =0.62; α_{CI} =0.10 and α_{BCI} =0.28. Figure 3.3 shows the relative weights of the three indices used in this study. For *IFI* and *BCI*, the higher the value the more attention is required for the bridge. On the other hand, the bridge with the lowest cost will get the highest priority amongst others. After normalizing the values of *IFI*, *CI* and *BCI* obtained in Tables 3.14, 3.16 and 3.17, the normalized decision matrix was obtained (Table 3.19). In order to convert the problem to a maximization problem the total cost is converted to benefit. After multiplying the normalized values with the weighting factor associated with each index and summing them up the *BPI* of each bridge is obtained (Table 3.19). Finally, the bridges are ranked as per their *BPI*. The higher the *BPI* values, the more attention is required for that particular bridge. The bridge authorities can refer to this *BPI* and the approved budget to select specific bridges for maintenance and rehabilitation. In this case the cardinal ranking of the bridges are **B4, B5, B1, B3, B2, B10, B7, B6, B9 and B8.**

3.5 SENSITIVITY ANALYSIS

Although the pair-wise comparison showed a possible scenario where, for example, *IFI* is clearly the most important criteria (see Figure 3.3), the AHP solution might change in accordance with shifts in analyst opinion. The final rankings of the competing alternatives are highly dependent on the weights attached to the main criteria. Small variations in the relative weights can result in a major change in the final ranking. Since these weights are usually derived from pair-wise comparison and based on highly subjective judgments, the stability of the ranking under varying criteria weights needs to be tested. Sensitivity analysis is necessary to investigate the robustness of model solutions. Criteria weights can have significant impact on the final decision making process. Sensitivity analysis by varying the priority (relative importance) of weights can be an effective mechanism to investigate this. The objective is to assure whether a few alteration in the judgement evaluations can lead to significant modifications in the final ranking. If the ranking is highly sensitive to small changes in the criteria weights, a careful revision of the weights is necessary. For this reason, sensitivity analysis is carried out to investigate the sensitivity of the alternatives by varying the priorities of the criteria at the level immediately below the goal. While changing the priority of one criterion, the priorities of the remaining criteria must be altered accordingly. After changing the priorities of all the attributes, the global priorities of the alternatives must be re-evaluated. For this purpose, the weights of the three criteria are separately altered and the results are reported in Figure 3.4(the weights of the other criteria are changed accordingly reflecting the relative nature of the weights, i.e., the total weights has to add up to 1).

Table 3.18: Pair-wise comparison matrix for three main criteria

Criteria	***IFI***	***CI***	***BCI***	**Priority vector**
IFI	1.000	5.000	3.000	0.62
CI	0.200	1.000	0.250	0.10
BCI	0.333	4.000	1.000	0.28

λ_{max}= 3.09; IC= 0.045; CR=0.077

Table 3.19: Sensitivity analysis result (scenario-I)

α	**0.62**	**0.10**	**0.28**		
Bridge No	*IFI*	*CI*	*BCI*	*BPI*	Rank
B1	0.122	0.142	0.124	0.125	3
B2	0.112	0.039	0.079	0.096	5
B3	0.112	0.081	0.101	0.106	4
B4	0.136	0.314	0.132	0.152	1
B5	0.126	0.183	0.118	0.130	2
B6	0.074	0.043	0.094	0.077	8
B7	0.078	0.043	0.099	0.080	7
B8	0.082	0.027	0.072	0.074	10
B9	0.084	0.028	0.077	0.076	9
B10	0.075	0.099	0.103	0.085	6

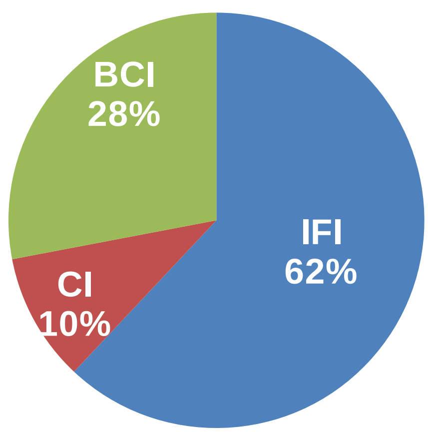


Figure 3.3: Relative importance of three main indices

The sensitivity graph (Figure 3.5) displays how the rank of the alternatives changes with respect to a change in priority of the criteria. Here. Scenario-I is the base case where the weights are obtained from AHP. In Scenario-II all the criteria have equal importance. In scenario-III the weight of *CI* is increased up to 100% (0.1 to 0.2) (from Scenario-I) while the weights of the other two criteria are reduced proportionately. Figure 3.5 portrays the ranking obtained in scenario-II where B4 remains the prime candidate with most of the ranking remaining unchanged. In scenario-IV the *IFI* is decreased by 50% (0.62 to 0.31) and the results indicated that the alternatives' ratings are not sensitive to changes in the importance of the criteria as B4 remains the first candidate to be prioritized. The sensitivity analyses results indicated that when the importance of the main criteria was changed up and down, the ranks of the alternatives remained almost stable in all cases. These analyses results show the robustness of the *BPI* which has less sensitivity to the criteria weights as B4 remains the first selection while increasing or decreasing the priorities of each criterion. Summarizing the results discussed above, one can affirm that the AHP method can be accepted with a good degree of confidence by the decision makers to prioritize bridges for fund allocation.

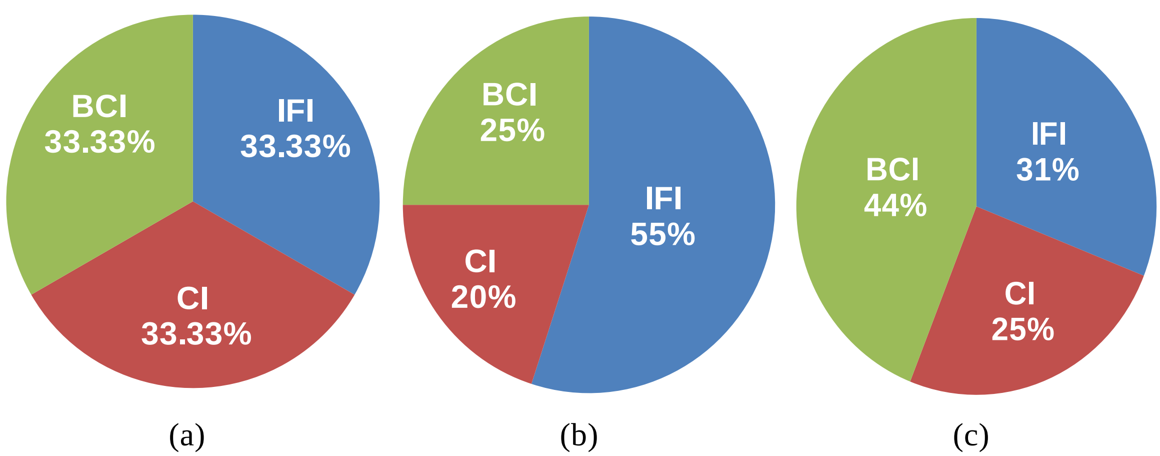
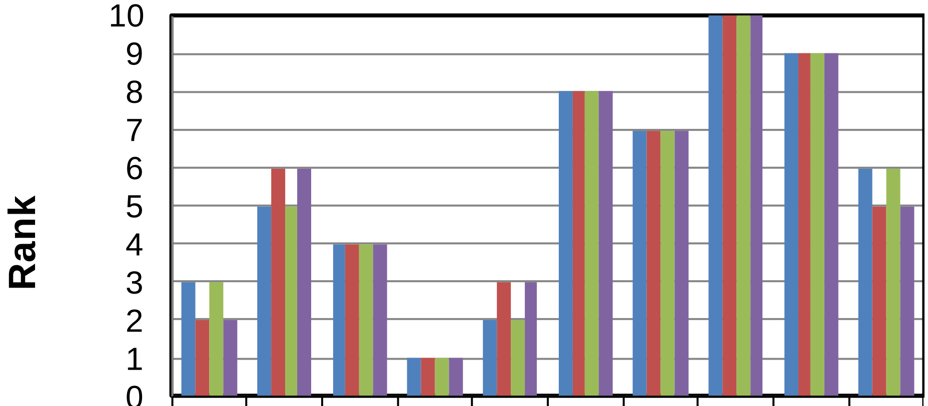


Figure 3.4: Relative importance of three indices (a) Scenario-II, (b) Scenario-III and (c) Scenario-IV



	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10
Scenario-I	3	5	4	1	2	8	7	10	9	6
Scenario-II	2	6	4	1	3	8	7	10	9	5
Scenario-III	3	5	4	1	2	8	7	10	9	6
Scenario-IV	2	6	4	1	3	8	7	10	9	5

Figure 3.5: Results of sensitivity analysis

3.6 SUMMARY

This study proposes an innovative ranking method for bridge networks, based on the AHP. The study also presents an innovative technique to develop the *BPI*. A prime issue involved in this proposed prioritization method is the visual inspection and inventory, not only on the structure, but also on the adjoining areas and facilities. This study also carried out sensitivity analysis for the priorities of the three “first-level” criteria and by changing criteria weights one at a time. This method does consider the main effects but not the interactions, which might be a little complex for real life application. The sensitivity analysis results prove the robustness of the proposed method.

CHAPTER 4 : SEISMIC PERFORMANCE EVALUATION OF RETROFITTED MULTI-COLUMN BRIDGE BENTS

4.1 GENERAL

Highway bridges play an important role to build a smooth and fast communication system in between cities and across country. Many existing bridges in North America may be inadequate in respect to the seismic performance required by the current codes and guidelines (ATC-32, CSA-2010, and ATC-49). Many of them were designed without any earthquake resistance criterion, because they were built prior to earthquake resistant design codes; others were designed to resist horizontal actions but without the principles of the capacity design or are built at a site in an area where the seismic hazard has been re-evaluated and increased (CSA-2010, ATC-49). The replacement or demolition of these bridges will be a costly undertaking. Alternatively, retrofitting of these bridges could be more convenient to meet current seismic and traffic demand. Various rehabilitation techniques are available to upgrade the seismic performance of existing RC structures. The major techniques for structural rehabilitation of RC bridges include encasing columns and beam column joints with steel, Fiber Reinforced Polymer (FRP) or reinforced concrete (RC) jackets or by adding new structural element. In recent years, innovative technologies such as Engineered Cementitious Composites (ECC) jacketing, prestressing wires, etc. along with traditional solutions, have become available to the practitioners for structural retrofitting by either enhancing the seismic capacity or reducing the demand. These options may be significantly different with respect to various aspects such as costs, time, structural performances, architectural impact, occupancy disruption, etc (FHWA-HRT-06-032, 2006, Priestely et al.,1994). In the last two decades, fiber reinforced polymers (FRP) have attracted the attention of researchers and bridge owners as an alternative material for retrofitting reinforced concrete bridge elements. The use of Engineered Cementitious Composites (ECC), a high performance high strength concrete with its special tensile properties have attracted the engineers to apply ECC where their properties can be beneficial, such as in earthquake engineering. Because of its improved tensile and strength properties, ECC warrants its use in seismic retrofitting.

Common deficiencies found in bridge bents built prior to 1965 are insufficient transverse reinforcement and inadequate lap splice length. In addition, poor detailing including poor anchorage of the transverse reinforcement, rare use of ties, and lap splices located in potential flexural hinge regions make older columns susceptible to failure. Possible failure modes of deficient columns are shear failure, pre-mature flexural failure and lap splice failure.

Inelastic static pushover analysis is a simple yet effective technique for assessing the strength capacity of structures in the post elastic range. Many seismic design guidelines (FEMA-356, ATC-40, and Eurocode-8) recommend pushover analysis as an effective tool for design and assessment of structures under seismic action. The ability of static pushover analysis (*SPO*) for assessing seismic performance of structures is well documented (Saiidi and Sozen, 1981; Fajfar and Gaspersic, 1996 and Bracci et al. 1999). Being a monotonic analysis, static analysis does not require unloading and reloading models. Although these procedures are much simpler than their dynamic counterpart, such techniques are not able to take into account progressive structural stiffness degradation, change of modal characteristics and period elongation of the structure for increasing values of external action (Ferracuti et al. 2009). Incremental Dynamic Analysis (*IDA*) is capable of accurately predicting the seismic capacity and demand of structures from elastic to dynamic instability ranges, by using a series of non-linear ground motion time-histories of increasing intensity (Vamvatsikos and Cornell, 2002; Mwafy and Elnashai, 2001). It can be noted that similar material constitutive relationship can be used in both static and dynamic analyses with the exception that static analysis does not require unloading and reloading models (Mwafy and Elnashai, 2001).

The objective of this research is to compare the performance of a pre-1965 designed multi column bridge bent retrofitted with different rehabilitation techniques, namely FRP jacketing, steel jacketing, concrete jacketing and engineered cementitious composites (ECC) jacketing. Moreover, the study aims to develop dynamic pushover envelopes from *IDA* for bridge bents retrofitted with different techniques. This study also investigates the performance of the retrofitted bridge bents with different analysis techniques i.e. *SPO* and *IDA*. A statistical analysis of the results obtained from *SPO* and *IDA* has been carried out to assess the ability of the static analysis in predicting the results obtained from the dynamic analysis. An interesting comparison in terms of capability of the pushover method to predict the performance points attained in the dynamic analyses has also been presented.

4.2 BRIDGE BENT DETAILS

To evaluate the performance of the retrofitted multi-column bridge bent, the northbound lanes of the South Temple Bridge is considered in this study (Pantelides and Gergely, 2002). The bridge was considered as seismically deficient as it had inadequacy in the amount of reinforcement and seismic detailing. This bridge bent was retrofitted by Pantelides and Gergely(2002) using CFRP jacketing. They developed design equations for CFRP jacketing and performed both experimental study and analytical verification of their results. The bent consists of three columns and a bent cap, as shown in Figure 4.1. A concrete deck of 21.87 m

(71.73 ft) span was supported by two bents and each bent supported eight steel girders. A gravity load of 240 kN (53.95 kip) was carried by each steel girder. Reinforcement details of the column, bent cap, and joints are shown in Figure 4.2. The bent column had inadequate transverse reinforcement in the lap-splice region. Transverse hoops in the bent cap joints were absent, and columns had insufficient tie spacing in the plastic hinge regions which is the most vulnerable portion of a column. The reinforcing steel in the bridge bent had a yield strength of 275 MPa (40 ksi), while the compressive strength of the concrete was 21 MPa (3 ksi).

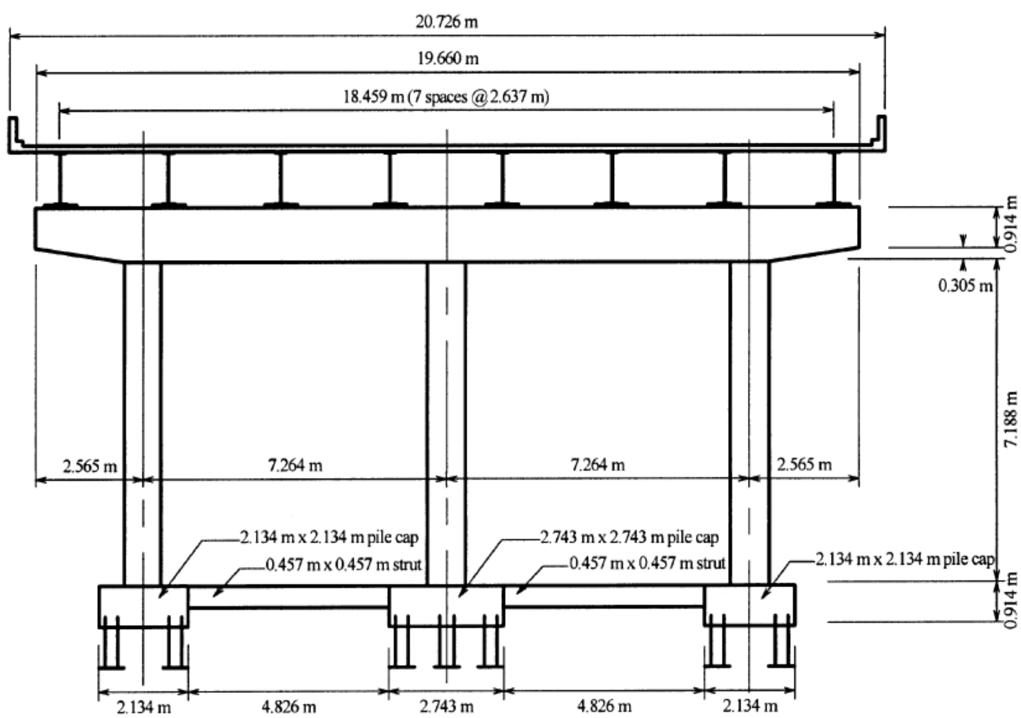


Figure 4.1: South Temple Bridge bent dimensions (Pantelides and Gergely, 2002)

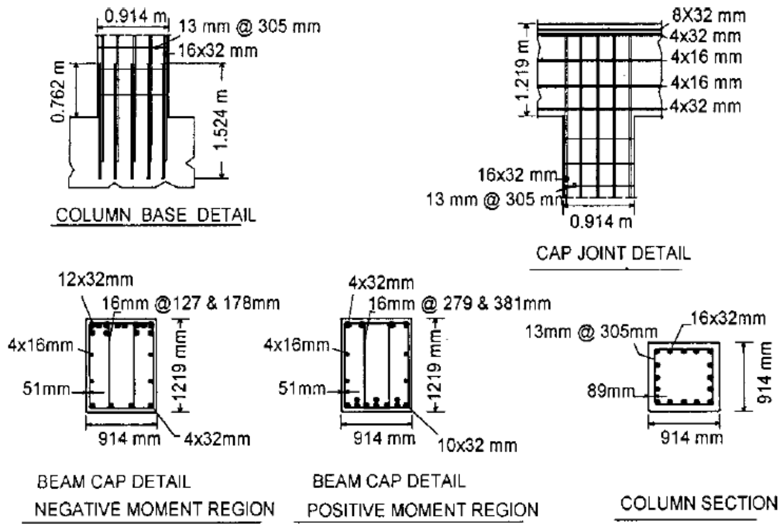


Figure 4.2: South Temple Bridge bent reinforcement details (Pantelides and Gergely, 2002)

4.2.1 Details of Retrofitting Techniques

Four different retrofit techniques specifically concrete jacketing, steel jacketing, CFRP jacketing and ECC jacketing have been implemented here to retrofit the seismically deficient multi column bridge bent. The mechanical properties of different retrofitting materials are provided in Table 4.1. In order to design the four different retrofitting techniques, a response spectrum analysis was carried out to determine the design base shear. As the bridge bent was located in Salt Lake City, Utah, USA, the design response spectrum for this location was obtained and shown in Figure 4.3. Determining the time period and modal mass participation factors from eigenvalue analysis, the design base shear for each retrofitted bridge bent was calculated using the square root sum of square (SRSS) method.

In this study the CFRP composite jacket retrofitting technique has been implemented from Pantelides and Gergely (2002). The material is a carbon fiber/epoxy resin composite with 48,000 fibers per tow unidirectional carbon fibers. The number of tows per 25.4 mm (1 inch) of sheet (pitch) was 6.5, and the width of the carbon fiber sheets was 457 mm (18 inch). The properties of the ambient temperature-cured CFRP composite were determined according to the ASTM D 3039 specifications (ASTM 1996). The thickness of the CFRP jacketing was calculated to be 3.42 mm.

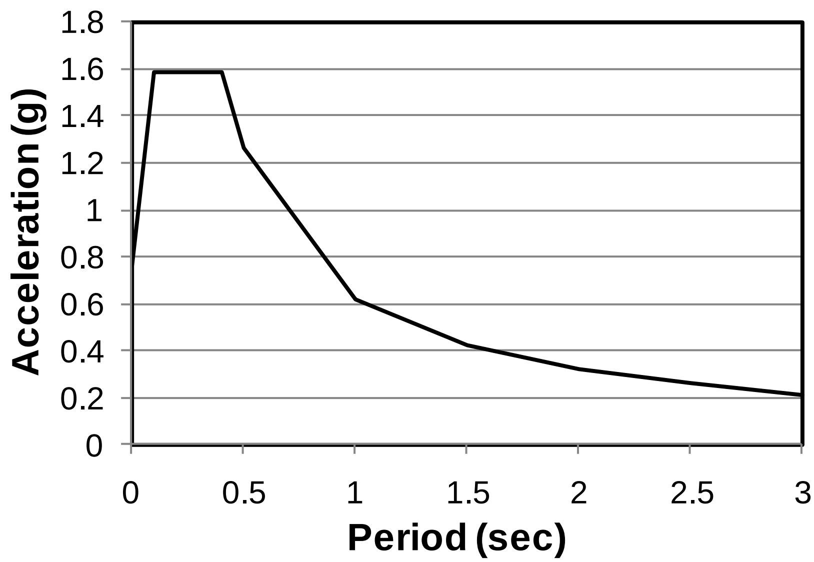


Figure 4.3: Design acceleration response spectrum

Elliptical steel jacket has also been used in this study which has been found effective for retrofitting of rectangular reinforced bridges columns. The steel jackets are positioned over the portion of the column to be retrofitted, and the vertical seams are then welded. The gap between the jacket and the column is grouted with a pure cement grout, after flushing with water. At the end of the jacket, a 50mm (2in.) gap is necessary between the supporting member and the jacket, which is grouted with cement mortar to avoid an increase in the stiffness at the plastic hinge region, which will attract greater internal forces that can be transferred to footing or cap beams.

Significant increases in flexural strength are possible from this source, which may then result in undesirable overload of the adjacent member (FHWA-HRT-06-032, 2006). The dimensions and thickness of the elliptical jackets are calculated according to FHWA-HRT-06-032 (2006). Smooth setting of the steel jacket and availability of structural steel plates restricts the thickness of steel jacket. The thickness required for steel jacket was 10 mm. The yield strength for steel jacket is considered as 400 MPa (58 ksi).

Table 4.1: Material properties for various retrofit alternatives

Material	Property	
CFRP	Tensile strength (MPa)	628
	Modulus of Elasticity (GPa)	65
	Ultimate axial strain (mm/m	10
Steel	Yield Strength (MPa)	400
	Modulus of Elasticity (GPa)	200
Concrete	Compressive strength (MPa)	34
	Strain at peak stress	0.002
ECC	Compressive strength (MPa)	80
	Tensile strength (MPa)	6.5

Concrete jacketing has been considered as another potential technique for retrofitting the deficient multi-column bridge bent. Applying full or partial height concrete overlays to the face of an existing column can increase a column's flexural strength. The new section must be well connected with the older one. Usually a jacket thickness of 90-150 mm is used. In this study the jacket thickness calculated was 120mm. This thickness is required to provide sufficient cover to the perimeter tie and to allow forming 135 hooks at the ends of the stirrups. Due to its superior property, ECC jacketing has been utilized as another retrofitting technique in this study.

Because of the strain-hardening property of ECC, this ductile material behaves more like steel than traditional concrete. The jacket thickness calculated was 80 mm.

4.3 FINITE ELEMENT MODELING

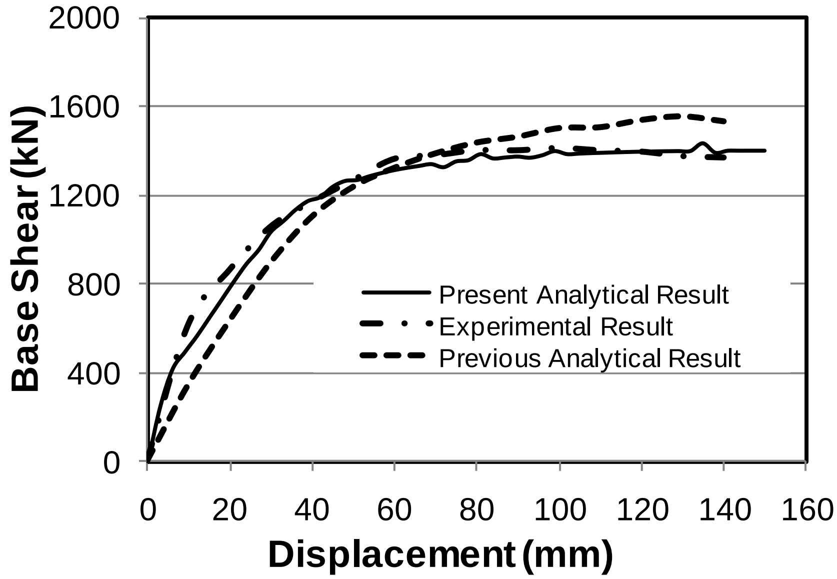
The analytical model of the bridge bent is approximated as a continuous 2-D finite element frame using the SeismoStruct nonlinear analysis program (SeismoStruct, 2010). Nonlinear static pushover and incremental dynamic time-history analyses have been performed on the bridge bents to determine the performances of the retrofitted bridge bents. The program has the ability to figure out the large displacement behaviour and the collapse load of framed structures accurately under either static or dynamic loading, while taking into account both geometric nonlinearities and material inelasticity (Pinho et al. 2007). 3D inelastic beam elements have been used for modeling the beam and the columns. The fibre modeling approach has been employed to represent the distribution of the material nonlinearity along the length and cross-sectional area of the member. Each fiber has a stress–strain relationship, which can be specified to represent unconfined concrete, confined concrete, and longitudinal steel reinforcement. The confinement effect of the concrete section is considered on the basis of reinforcement detailing. The distribution of inelastic deformation and forces is sampled by specifying cross-section slices along the length of the element.

To develop the analytical model Menegotto-Pinto steel model (Menegotto and Pinto, 1973) with Filippou (Filippou et al., 1983) isotropic strain hardening property is used for reinforcing steel material. The yield strength, strain hardening parameter and modulus of elasticity of steel are considered as 275MPa (40 ksi), 0.5% and $2x10^5$MPa (29000 ksi), respectively. For concrete non linear variable confinement model of Madas and Elnashai (1992) with compressive strength of 21MPa and tensile strength 1.7 MPa has been used. CFRP has been modeled using non linear

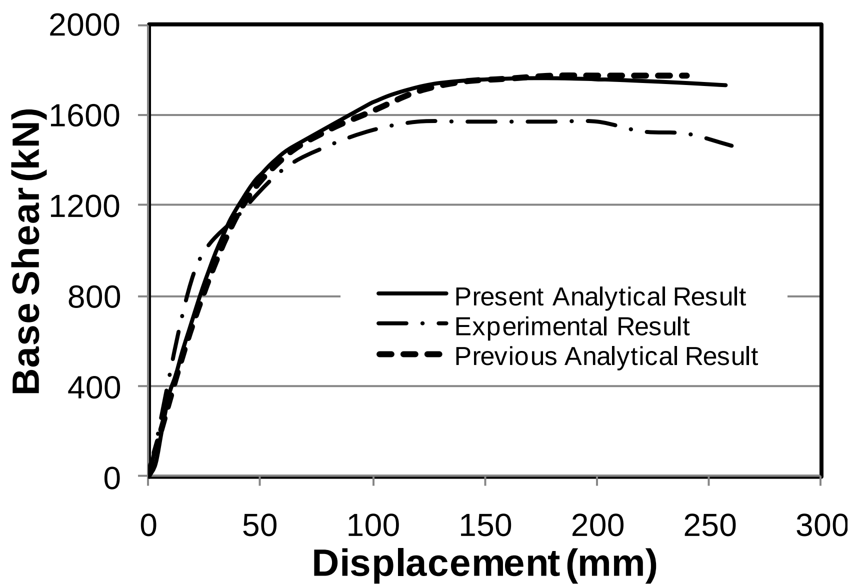
FRP confined concrete model developed by Ferracuti and Savoia (2005). For compression, this model follows the constitutive relationship and cyclic rules proposed by Mander et al. (1988), and those of Yankelevsky and Reinhardt (1989), for tension. FRP confined concrete model proposed by Spoelstra and Monti (1999) have been employed to model the effects of the confinement introduced by the FRP wrapping. The retrofitted parts of the bridge bent have been modeled in Seismostruct with jacketed section. To develop the analytical model for bridge bent retrofitted with ECC jacket, another finite element software ZeusNL (2011) was employed. Similar concrete and steel model was used for modeling bridge bent and ECC jacket was modeled following the constitutive model developed by Han et al. (2003)

4.4 NONLINEAR STATIC PUSHOVER ANALYSIS

Nonlinear static pushover analysis has been performed for each bridge bent considering a 2-D frame using SeismoStruct (2010). The girder load of 240 kN (53.95 kip) is applied as a permanent load at each girder location and for the pushover analysis incremental load is applied in the form of displacement up to a magnitude of 0.3 m. Figure 4.4a illustrates the comparative study of pushover response curve for the previous experimental and analytical result on the south temple bridge bent along with the results obtained from the present analytical model. The present analysis provided better results compared to those of previous analytical results (Pantelides and Gergely, 2002) and could simulate the initial stiffness, post elastic stiffness, and ultimate load carrying capacity accurately when compare with the test results. Figure 4.4b depicts the comparative performance of CFRP jacketed bridge bent for the previous experimental and analytical result (Pantelides and Gergely, 2002) with the current analytical model. From Figure 4.4b it is evident that the present analytical result is very close to the previous analytical result.



(a)



(b)

Figure 4.4: Pushover response curve for bridge bent (a) comparison with previous study as built, (b) comparison with previous study retrofitted with CFRP

The pushover response curves for the different retrofitting techniques namely steel jacketing, concrete jacketing, CFRP jacketing, ECC jacketing and the original bridge bent are shown in Figure 4.5. From the pushover response curves it is observed that the different retrofitting techniques increased the lateral capacity of the bridge bent considerably.

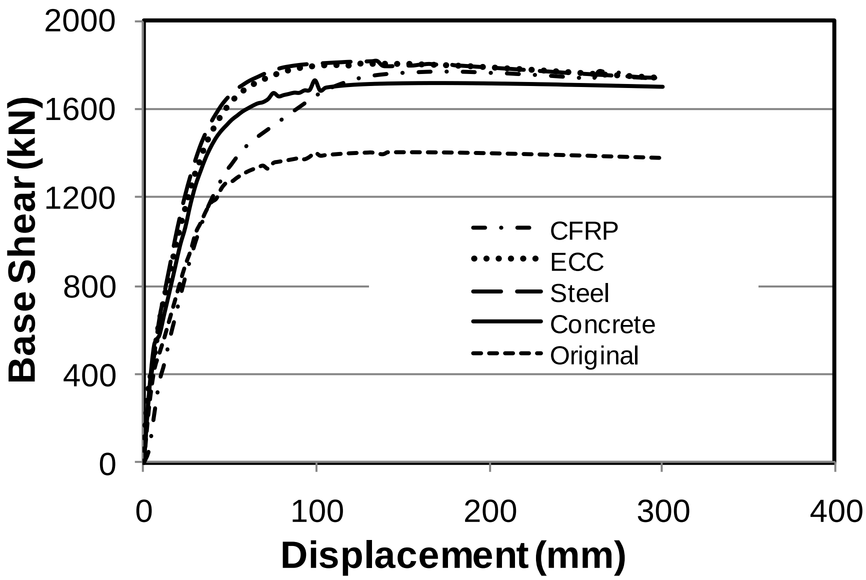


Figure 4.5: Pushover curves for bridge bent retrofitted with different alternatives

From the pushover analysis it is observed that the steel jacket increased the capacity most while concrete jacket was on the lower side. However, in terms of capacity all four retrofitting techniques showed similar performance. Figure 4.6 presents the ductility for bridge bent retrofitted with different retrofitting techniques. Ductility is defined as the ratio of the bent top displacement at maximum load to the bent top displacement at yield load. The maximum value of the ductility is found for steel jacketed bridge bent which was 3.96. Bridge bent retrofitted with Concrete jacket showed lower ductility as compared to the other three techniques. ECC jacketed bridge bent showed higher ductility as compared to normal concrete and CFRP jacket.

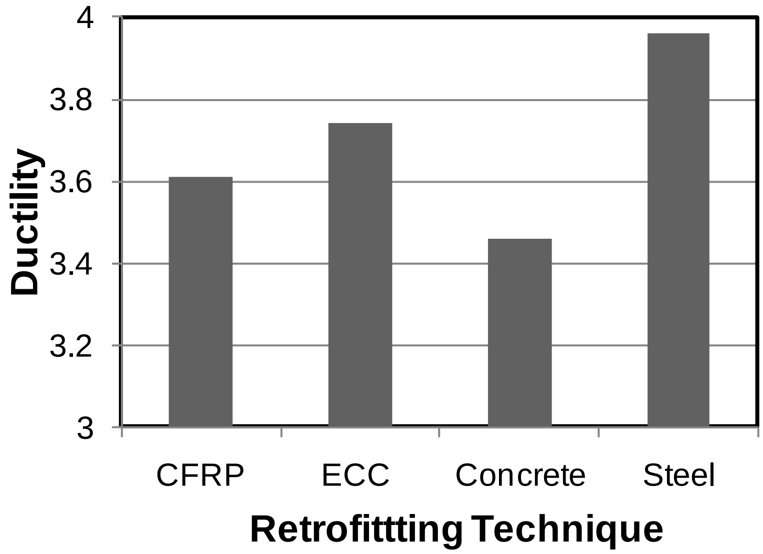


Figure 4.6: Ductility for bridge bent

The four retrofitting techniques considered in this study are compared in terms of different performance criteria. The performance criteria considered here are the displacements and base shear at the onset of concrete cracking, yielding of longitudinal steel and crushing of concrete. Table 4.2 shows the comparative performance of different retrofitting techniques. From Table 4.2 it is evident that cracking started at all the retrofitted bridge bent almost at the same level of displacement. The base shear at cracking for steel jacketed section was 520.5 kN which was 20%, 14% and 7% higher than that of CFRP, concrete and ECC jacketed section, respectively. On the other hand, the CFRP retrofitted bridge bent underwent large displacement before yielding which was 27% higher than that of steel jacketed section, where the steel jacketed section encountered higher amount of base shear which was 5% higher than that of CFRP jacketed section. Before the crushing of the concrete, steel jacketed bridge bent suffered less displacement while encountering larger base shear which was 1%, 3% and 4% higher than that of the ECC, concrete and CFRP jacketed bridge bents, respectively.

Table 4.2: Comparative performance of different retrofit techniques from *SPO*

	Concrete Cracking		**Steel Yielding**		**Concrete Crushing**	
Retrofit option	Displacement (mm)	Base Shear (kN)	Displacement (mm)	Base Shear (kN)	Displacement (mm)	Base Shear (kN)
CFRP	5.94	416	32.68	1263	60.81	1647
Concrete	5.95	448	29.87	1298	54.83	1664
Steel	5.95	520.5	23.88	1335	48.85	1718
ECC	6.58	481.7	30.47	1302	57.83	1698

4.5 INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis (*IDA*) technique was developed by Luco and Cornell (1998) and has been described in detail in Vamvatsikos and Cornell (2001) and Yun et al. (2002). *IDA* is a parametric method, which requires a series of nonlinear dynamic analyses of a modeled structure for an ensemble of ground motions of increasing intensity with the objective of attaining an accurate indication of the nonlinear dynamic response of the structure under an earthquake action. Intensity levels are selected to cover the entire range of structural response, from elastic behaviour through yielding to dynamic instability (or until a limit state ‘‘failure’’ occurs). From the results of these multiple analyses, statistics on the variation of demand and capacity with ground motion characteristics can be evaluated to summarize the results. Seismic demands of mid-rise buildings can be determined with adequate accuracy if 10-20 ground motions are employed for *IDA* (Shome and Cornell, 1999). In this study ten selected earthquake records have been used for incremental dynamic analyses as shown in Table 4.3**.** The records selected belong to a bin of relatively large magnitudes, 6.5–6.9, and moderate epicentral distances in the range of 15–32 km. The records are obtained from the PEER strong motion database. Figure 4.7 shows the acceleration response spectrum (5% damped) for the selected ground motion sets.

Table 4.3: Selected earthquake ground motion records

No	Event	Year	Record Station	Φ^1	M^{*2}	R^{*3} (km)	*PGA* (g)
1	Imperial Valley	1979	Plaster City	45	6.5	31.7	0.042
2	Imperial Valley	1979	Plaster City	135	6.5	31.7	0.057
3	Imperial Valley	1979	Cucapah	0	6.9	16.9	0.309
4	Imperial Valley	1979	El Centro Array#13	140	6.5	21.9	0.117
5	Imperial Valley	1979	El Centro Array#13	230	6.5	21.9	0.139
6	Loma Prieta	1989	Coyote Lake Dam	285	6.5	22.3	0.179
7	Loma Prieta	1989	Agnews state hospital	90	6.9	28.2	0.159
8	Loma Prieta	1989	Sunnyvale Colton Ave	270	6.9	28.8	0.207
9	Loma Prieta	1989	WAHO	0	6.9	16.9	0.37
10	Superstition Hill	1987	Wildlife liquefaction array	90	6.7	24.4	0.18

^{1}Component, ^{2}Moment Magnitudes, ^{3}Closest Distances to Fault Rupture
Source: PEER Strong Motion Database, http://peer.berkeley.edu/svbin

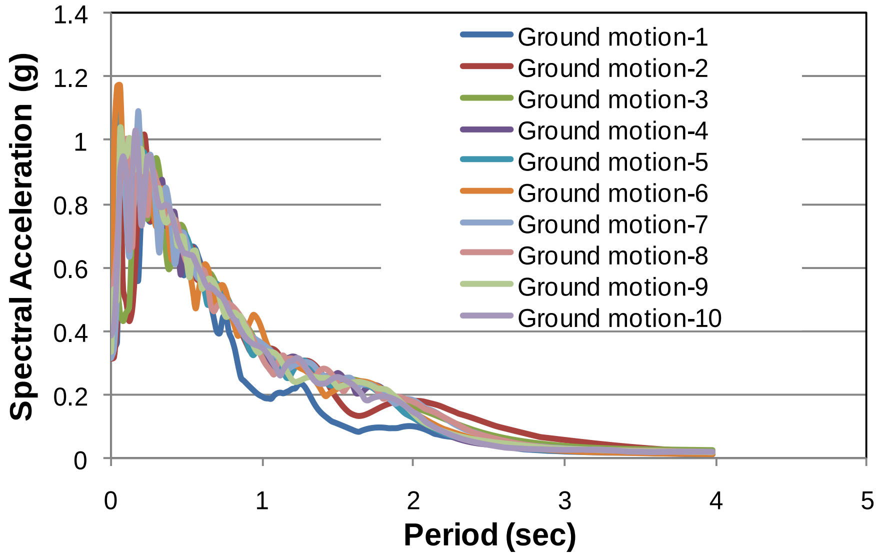


Figure 4.7: Spectral acceleration for the chosen earthquake records

4.5.1 Performance Criteria and Incremental Dynamic Analysis

Once the model is generated and the ground motion records are chosen, *IDA* is performed. Thus, a nonlinear computational model of the prototype structural system was developed. To start the analysis, the chosen earthquake records need to be scaled from a low intensity measure (*IM)* to several higher *IM* levels until structural collapse occurs. The criteria used for comparing the performances are the displacements and the base shear at the onset of concrete cracking, yielding of longitudinal steel and crushing of concrete. Figure 4.8 shows the box plot of data obtained from *IDA* for four different retrofitting alternatives. In each figure, individual retrofitting technique has a diagram where the height represents the numerical range of the data (maximum and minimum values) for displacement and base shear at yielding and crushing. The "boxes" represent the 25th through the 75th percentile. The horizontal line inside the box is the median value (or 50th percentile). Several researchers (Lehman and Moehle, 2000; Otani, 1981) have identified that number of cycles and loading pattern have an impact on the yield and ultimate displacement and load carrying capacity of RC structures. Under seismic loading the structure experiences multiple reverse cyclic accelerations and with increasing intensity level the structure exhibits a wide range of yield and ultimate displacement. Figure 4.8 shows the range of different performance criteria obtained from the *IDA*.

For yield displacement CFRP jacketed bridge bent showed higher values, while that for steel jacketed bridge bent was on the lower side. The CFRP jacketed section showed standard deviation of 2.64, which was 27%, 16% and 17% higher than that of concrete, ECC and steel jacketed bridge bent, respectively. Bridge bent retrofitted with concrete jacketing encountered large variation of base shear at yielding (maximum 2100 kN and minimum 1302 kN) for different ground motions. CFRP jacket showed stable values of yielding base shear for different ground motions. For crushing displacement, steel jacketed section resulted in a standard deviation of 6.4 which was 3.2, 1.6 and 1.8 times higher than that of CFRP, Concrete and ECC jacketed bridge bent, respectively. At crushing of the core concrete, the CFRP jacketed bridge bent could sustain a maximum base shear of 2038 kN, which was the lowest of all four alternatives. On the other hand, steel jacketed section sustained a maximum base shear of 2525 kN.

From Figure 4.8 it is seen that for the different performance criteria, each retrofitted bridge bent displayed a range of values. This is attributed to the difference in loading cycle and magnitude of different earthquake records. From Figure 4.8 it is evident that different retrofit techniques exhibited different ranges for different performance criteria. This difference is mainly due to the difference in material properties and damage tolerance properties of different retrofitting materials under cyclic loading.

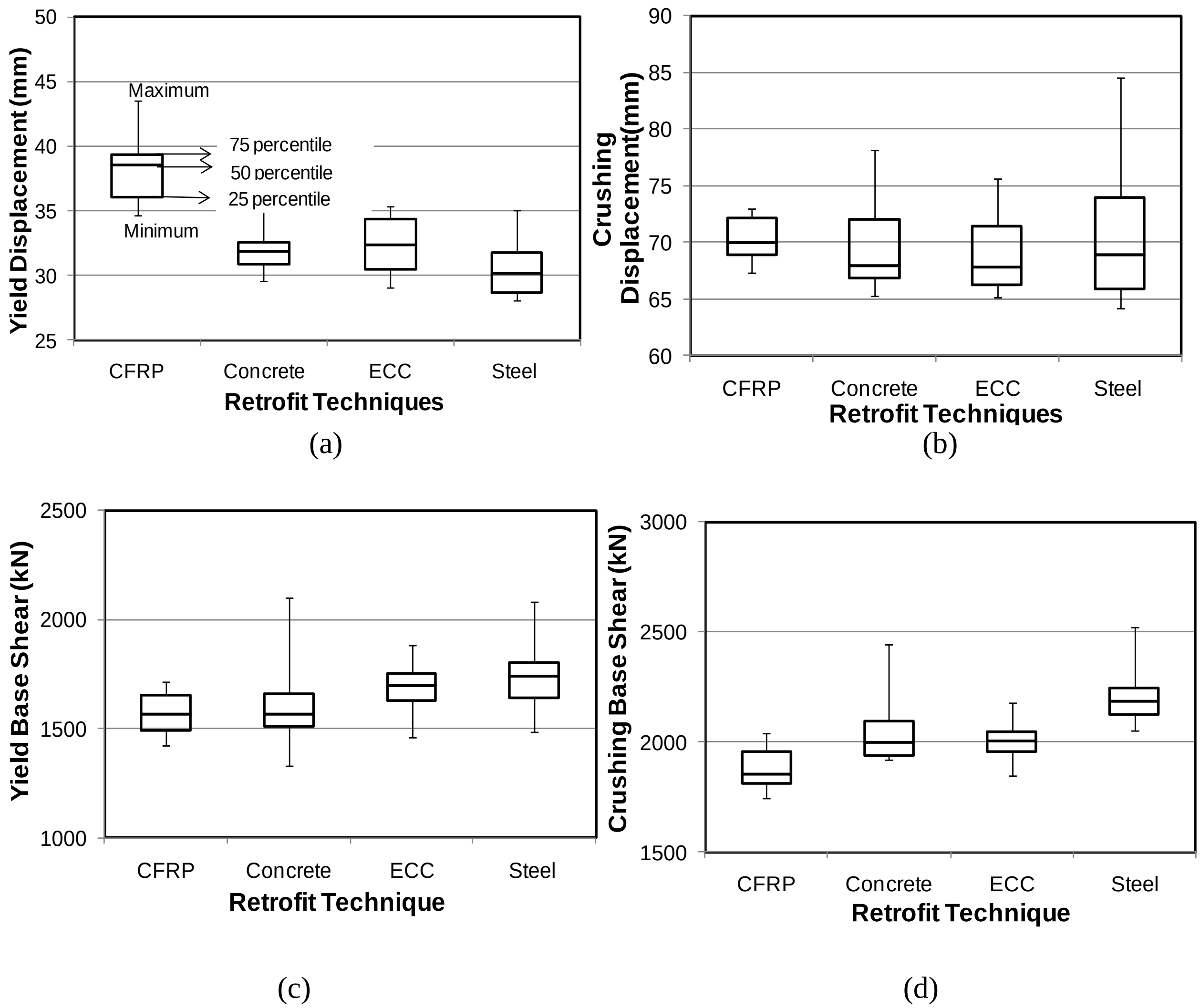


Figure 4.8: Box plot of data obtained from *IDA* for four retrofit alternatives (a) yield displacement, (b) crushing displacement, (c) yield base shear and (d) crushing base shear

4.5.2 Comparison of Different Retrofit Alternatives

This section presents the results of the dynamic responses of four retrofitted bridge bents under the Imperial valley earthquake (record no. 3 in Table 4.3).The performance of different retrofit alternatives are compared in terms of the base shear capacity demand ratio, residual drift percent, ductility and damage states to find out the most efficient retrofit option.

4.5.2.1 Base Shear Capacity/Demand Ratio

Figure 4.9 shows the base shear capacity-demand ratio of four different retrofit techniques. From this figure it is observed that the capacity-demand ratio in terms of base shear for the retrofitted bridge bents varies with different intensity levels. The higher the intensity, the higher is the demand, and consequently the lower is the capacity-demand ratio. At all intensity levels all the retrofitted bridge bents showed quite similar capacity-demand ratio except some cases where the bridge bent retrofitted with ECC jacketing showed higher capacity. This is attributed to the high tensile capacity of ECC. For instance, at spectral acceleration of 1g the ECC jacketed bent showed a capacity demand ratio of 3.75 where that of other three options were less than 2.5.

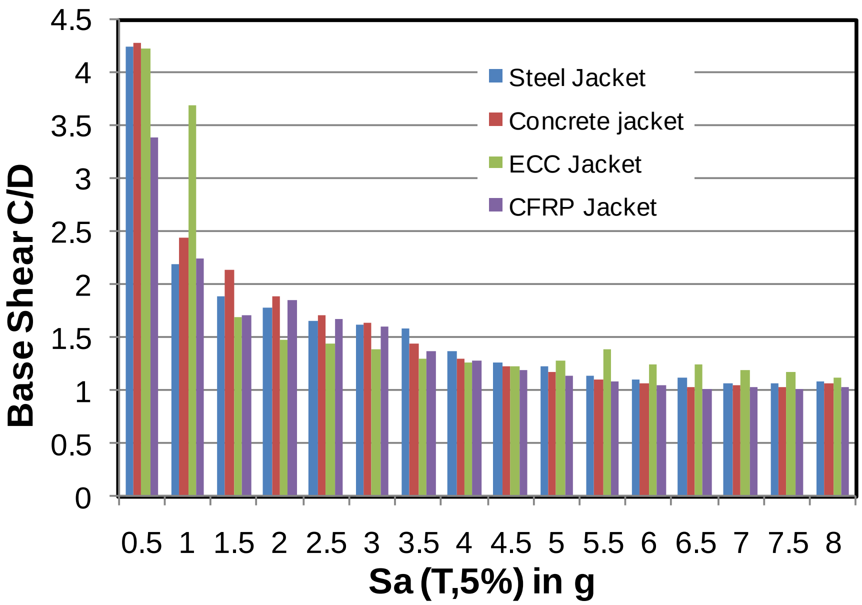


Figure 4.9: Bridge bent capacity-demand ratio in terms of base shear (ground motion-3)

4.5.2.2 Residual Drift Ratio

Residual drift is an important parameter for performance based earthquake engineering. A limited number of studies have been performed by a few researchers specifically with regard to predicting residual drift (Sakai et al., 2005 and Hachem et al., 2003). A comparison between the residual drift (%) sustained by the different retrofitted bridge bents is shown in Figure 4.10. There is a clear difference in the response of the four retrofitting systems. With increasing intensity, all the retrofitted bridge bents begin to sustain significant residual displacements, with large variation in the magnitudes of the residual displacements for the different intensities. The bridge bent retrofitted with ECC jacketing experienced the lowest residual drift with a maximum of only 1.45% as shown in Figure 4.10. On the other hand, the bridge bents retrofitted with steel and concrete jacketing had a maximum residual drift of 2.1% and 2.2%, respectively. Bridge bent retrofitted with CFRP jacketing experienced substantially a lower residual drift (maximum 1.66%) with increasing intensity, with much less variation in the results. When the residual drift ratio exceeds 1 percent, the functionality of a bridge becomes questionable (Lee and Billington, 2009). In this case all the retrofitted bridge bents have their residual drift over 1 percent at spectral acceleration over 6g. Both CFRP and ECC jacketed bridge bent experienced a residual drift of less than 1% up to spectral acceleration of 6g. This reveals the efficacy of both CFRP and ECC jacketing techniques.

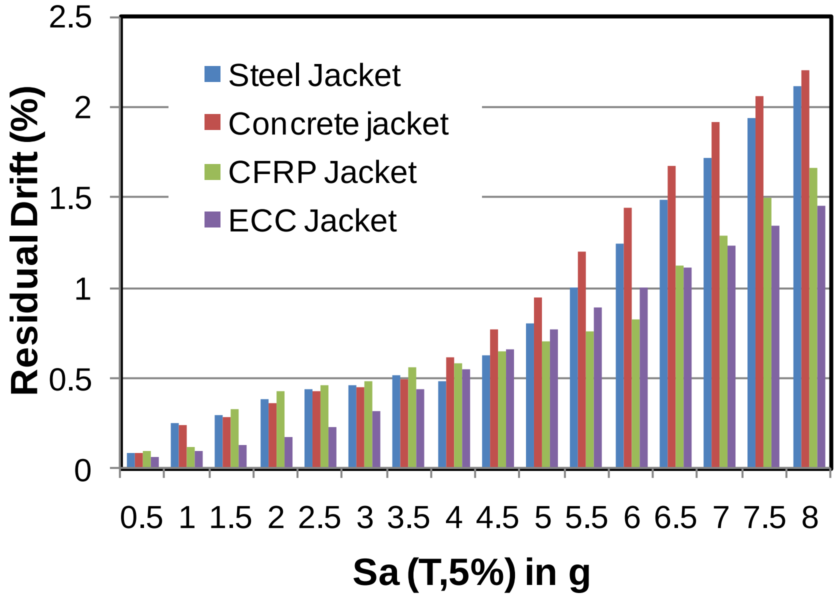


Figure 4.10: Residual drift (%) for different retrofit techniques (ground motion-3)

4.5.2.3 Ductility Demand

For analytical assessment purpose the limit states or damage states need to be defined in terms of some demand parameter. Damage states for bridges should be defined in such a way that each damage state indicates a particular level of bridge functionality. Ductility plays a significant role in evaluating the performance of a bridge structure during an earthquake event. It is defined as the ratio of ultimate displacement to the displacement corresponding to the yield strength. Ductility represents the energy dissipation capacity of the structure as well. Here, displacement ductility is used as a demand parameter to assess the performances of the four retrofitting technique. Figure 4.11 shows the ductility demand of the four retrofitted bridge bent plotted against different intensity level of ground motion-3. Different researchers have proposed different damage parameters for defining the bridge functionality limit states. In this study, the displacement ductility of the bridge pier is adopted as demand parameter. Dutta and Mander (1999) recommended five different damage states for bridge columns depending on drift limits.

Table 4.4 shows the damage states proposed by Dutta and Mander (1999). In this study, these drift limits are transformed to the ductility demand of the bridge pier for each limit state. Table 4.5 shows the values of ductility demand and the corresponding damage states for the four different retrofitted bridge bents. The ductility demands for various damage states for retrofitted bridge pier developed by Shinozuka et al. (2002) is also provided in table 4.5. It shows that the value obtained for steel jacketed pier closely matches with the values developed by Shinozuka et al. (2002).

Figure 4.11 reveals that ECC jacketed bridge bent did not suffer any damage up to spectral acceleration of 4g whereas the other three bents suffered slight damage. With increasing intensity the ductility demand increased significantly for all the retrofitting options. It is also evident from Figure 4.10 that after a certain intensity level all the retrofitted bridge bent except the one with ECC jacketing reached the extensive damage state. This numerical result revealed that ECC jacketing performed well in reducing the damage of the bridge bent. It is also evident from figure 4.10 that none of the retrofitted bents entered the collapse state.

Table 4.4: Damage/limit state of bridge components (Dutta and Mander, 1999)

Damage state	**Description**	**Drift limits**
Almost no	First yield	0.005
Slight	Cracking, spalling	0.007
Moderate	Loss of anchorage	0.015
Extensive	Incipient column collapse	0.025
Collapse	Column collapse	0.050

Table 4.5: Ductility demand of retrofitted bridge piers

Damage state	Retrofit Technique				
	CFRP Jacket	Concrete Jacket	Steel Jacket	ECC Jacket	Shinozuka et al. 2002 (Steel Jacket)
Almost no	1.0	1.0	1.0	1.0	1.0
Slight	1.62	1.81	2.25	1.72	1.80
Moderate	3.47	3.89	4.82	3.70	4.90
Extensive	5.78	6.48	8.04	6.16	8.90
Collapse	11.56	12.96	16.08	12.32	18.7

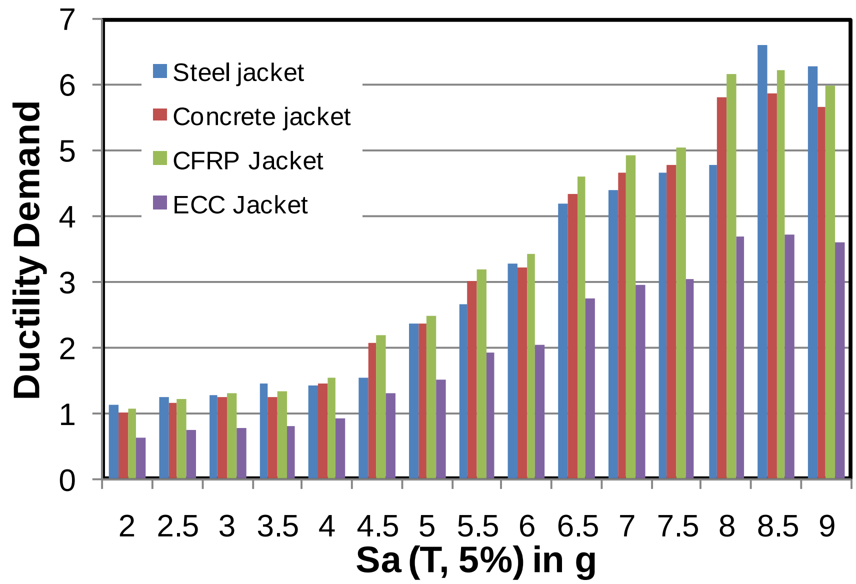


Figure 4.11: Ductility demand of different retrofit techniques (ground motion-3)

4.6 COMPARISON BETWEEN STATIC AND DYNAMIC PUSHOVER ANALYSIS

In this section, the nonlinear static pushover analysis responses of the retrofitted bridge bent with the four different techniques are compared with those of their incremental dynamic analysis counterpart. The results of more than 650 inelastic time history analyses were employed to perform the regression analyses to obtain the dynamic pushover envelope for each of the four different retrofitted bridge bent. Figure 4.12 depicts the dynamic response points and the fitted regression equations of the response of the bridge bents subjected to the ten seismic ground motions considered. The actual response of all four retrofitted bridge bents showed that the results of the ten seismic actions follow almost the same trend and shape of the pushover envelopes without the need to apply curve fitting. This is evident from the correlation coefficient values (R^2 values in Figure 4.12) which are almost close to 1.0. A number of capacity curve plots, obtained from the pushover analyses are compared with the *IDA* envelopes. Figure 4.12 also shows the comparison between the static and dynamic pushover curve for different retrofit options. It is to be noted here that the *SPO* and *IDA* are continued beyond all the predefined performance criteria point. This is to ensure that all the performance criteria are obtained during the dynamic analysis. From Figure 4.12 it is evident from the *IDA* curves that the *SPO* measured the capacity of the retrofitted bridge bents quite conservatively. Up to a certain level all the curves showed similar trend, and then the *IDA* curves showed higher capacity.

Figure 4.13 shows the difference in analysis results in terms of various performance indicators obtained from *SPO* and *IDA*. In this study, the performance points have been averaged for different intensity levels of each ground motion. In some cases the *SPO* overestimated the yield displacement of the retrofitted bridge bents by only few percentages (not more than 4%). But for all other cases the *SPO* underestimated the capacity of the bridge bent. This was more prominent especially in the case of estimating the crushing displacement. For steel jacketed bridge bent the capacity of the crushing displacement obtained from *IDA* was 40% higher than that from *SPO*, while for the other criteria it was less than 20%. For CFRP jacketed bridge bent, the estimated crushing displacement by the *SPO* was almost 30% less than that from the *IDA*. The *SPO* also underestimated the yield base shear by 20% on an average compared to the *IDA*.

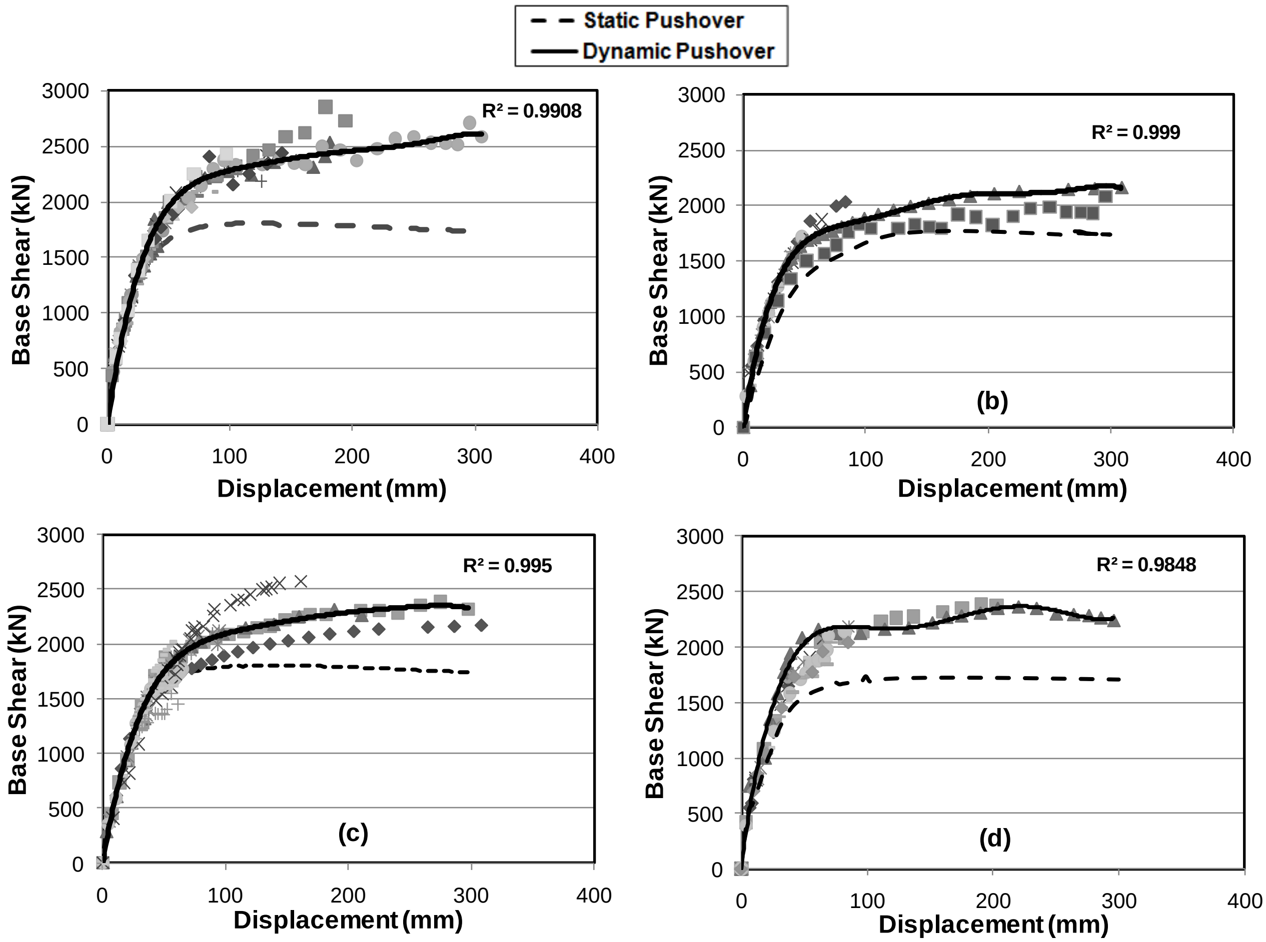


Figure 4.12: Static and dynamic pushover results for retrofitted bridge bents (a) steel jacket, (b) CFRP jacket, (c) ECC jacket and (d) concrete jacket

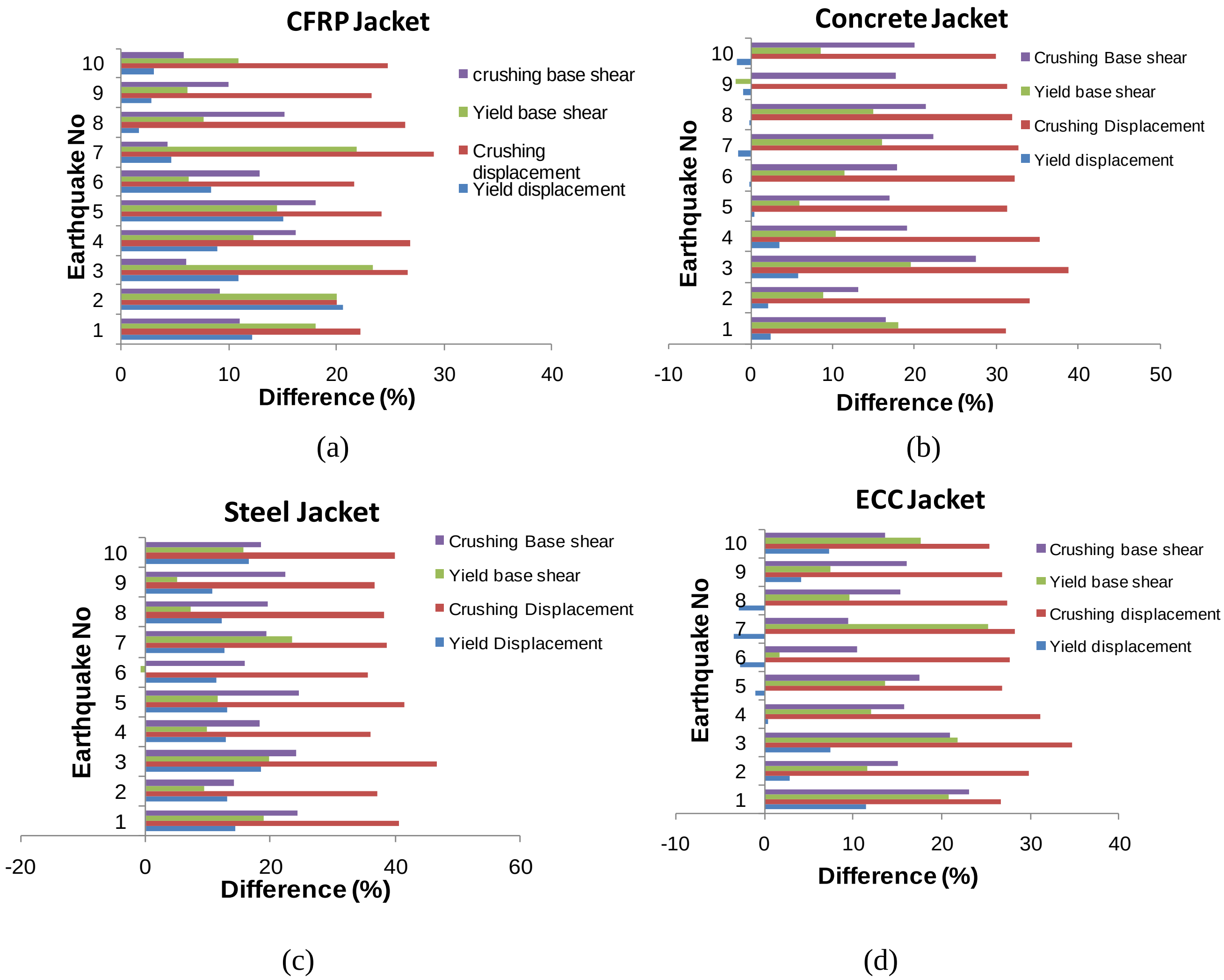


Figure 4.13: Difference between *SPO* and *IDA* at different performance criteria

Different values of the performance criteria considered in this study (displacements and base shear at the onset of concrete cracking, yielding of longitudinal steel and crushing of concrete) are obtained by *IDA*, depending on the considered time histories. Therefore, its variability has been described as a normal random variable, calculating mean value and standard deviation of the values of the performance points obtained from the different time-histories. Gaussian distributions of the displacements and the base shear corresponding to the performance points obtained from *IDA*s are shown in Figure 4.14. For conventional pushover analyses, displacements and base shear corresponding to performance points are (single) deterministic values, because these methods do not take the specific input motions into account. On the contrary, in the *IDA* procedure the input motion can be taken into account by employing, each time-history one at a time. Therefore, a statistical study has been performed on the results obtained from *IDA* only.

Figure 4.14 also depicts the single deterministic value (vertical lines) obtained from the *SPO* for each of the performance criteria for each retrofitting technique. Figure 4.14 also reveals that all the curves are negatively skewed which indicates that the mass of the distribution is concentrated on the right of the figure and the bulk of the value lie on the right of the mean. But the value obtained from the *SPO* is on the left side of the curves. For displacement at yield (Figure 4.14a), the values obtained from the *SPO* largely differ from the mean value obtained from the *IDA*, which ranges from 6% (ECC) to as high as 27% (Steel). Similar difference is also observed in the case of the crushing displacement (Figure 4.14b) where the difference in result obtained from the two analyses varies from 15% (CFRP) to 44% (Steel). Similar conclusion can be drawn by looking at the other two performance curves (Figure 4.14c, 4.14d) as well. Comparison of the mean value obtained from the *IDA* with the results obtained from the *SPO* reveals that the *SPO* procedure underestimates the performance points as predicted by the *IDA* by quite a good margin.

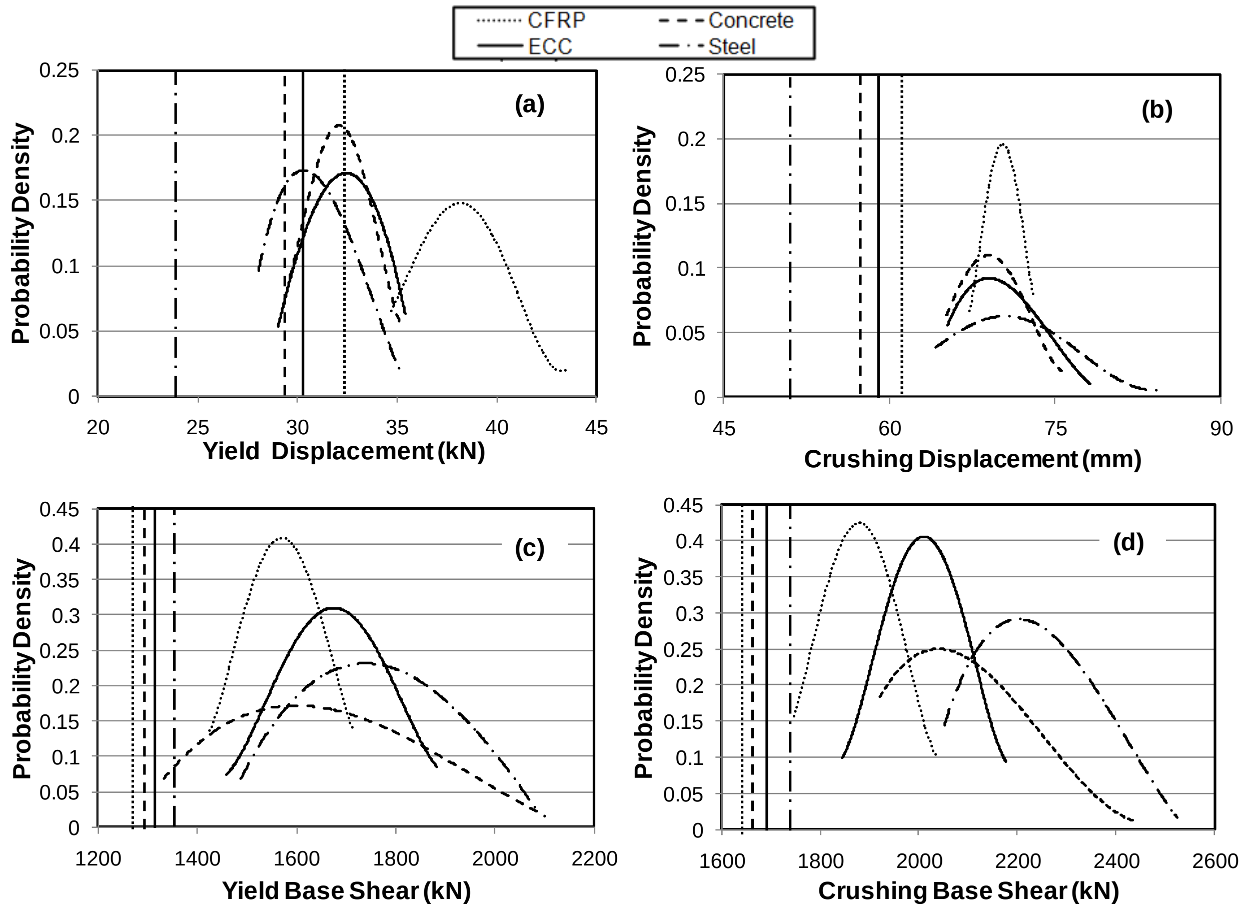


Figure 4.14: Probability density function of performance criteria from *IDA* and deterministic values (vertical lines) from *SPO* for retrofitted bridge bents

4.7 SUMMARY

Older bridges offer insufficient resistance to lateral excitation and fail to provide satisfactory performance during major seismic event. This study demonstrated the improvement in seismic performance brought about by the application of different retrofitting techniques. This study revealed the potential enhancement of seismic performance brought about by the use of ECC jacketing. The use of ECC resulted in reduced damage and residual displacement, which would substantially improve serviceability of bridge bents after strong earthquakes and lead to improved emergency response and economic recovery. This study demonstrated the inability of the *SPO* to accurately predict the seismic capacity of structures which often can be obtained through the *IDA*. Although limited by high computational cost and time, Incremental dynamic analysis is necessary to produce compelling evidence regarding seismic performance evaluation.

CHAPTER 5 : FRAGILITY ANALYSIS OF RETROFITTED MULTI-COLUMN BRIDGE BENT SUBJECTED TO NEAR FAULT AND FAR FIELD GROUND MOTION

5.1 GENERAL

Highway bridges are critical components of the overall transportation system as they play important roles in evacuation and emergency routes for rescues, first-aid, firefighting, medical services and transporting disaster commodities. In view of the importance of the bridge, it is a contemporary key issue to minimize the loss of the bridge functions as much as possible against earthquakes to enhance continued functioning of the community life. So, it is indispensable to evaluate the seismic vulnerability of bridges against earthquakes. The seismic vulnerability of highway bridges is often explicitly expressed in the form of fragility curves. Fragility curves indicate the conditional probability of a bridge sustaining a particular degree of damage when subject to a given level of ground shaking. Fragility-curve involves probabilistic seismic performance evaluation, which in turn provides the opportunity to make seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of highway systems.

Strong ground motions from major earthquakes near urban areas in recent years have demonstrated that near field ground motions are the most severe earthquake loading that structures undergo. The distinctive and catastrophic features of near field ground motions generally are not considered in seismic design although the peculiar structural response to near-fault ground motions has been documented (Bertero et al. 1978; Somerville et al. 1997). These near fault ground motions place serious demand on structures located in the near field region of an earthquake.

The objective of this chapter is to assess the fragility of a pre-1965 designed multi column bridge bent retrofitted with different rehabilitation techniques, namely CFRP jacketing, steel jacketing, concrete jacketing and engineered cementitious composites (ECC) jacketing, under near field and far field ground motions. The study aims to capture the impact of different retrofit techniques on the vulnerability of a retrofitted bridge bent, which to date has not been adequately addressed for both near and far field ground motions. In this regard, the analytical simulation method was used to evaluate the seismic fragility of the bridge bent. A 2-D finite element model scheme with nonlinear force-displacement relationships for the bridge bents were used in analytical modeling of the bridge. The seismic responses of the bridge bents for a total of 40 earthquake excitations of which 20 are near field and 20 are far field ground motions, are utilized to evaluate the likelihood of exceeding the seismic capacity of the bridge bent. Finally, the fragility functions are derived based on the simulation results from nonlinear time history analyses and are then combined to evaluate the overall fragility of the bridge bent.

5.2 NEAR AND FAR FIELD GROUND MOTION CHARACTERISTICS

Near field and far field ground motions differ from the distance to the rupturing fault line. According to Caltrans (2004) if the structure under consideration is within 10 miles (approximately 15 km) of a fault it can be classified as near-fault. Ground motions outside this range are classified as far-field motions. Somerville (2002) defined near fault ground motions are those, which often contain a long period velocity pulse and permanent ground displacement. Recordings from recent earthquakes indicated that the near field ground motion possesses some features such as distinctive pulse like time histories, high peak velocities and high ground displacement. Very often the near-fault motions result in a relative increase in the vertical component of the ground motion compared to the far-field motions (Xinle et al. 2007), which can have a devastating effect depending on the strength and characteristics of the ground motion.

Near fault ground motions possess some unique characteristics such as high *PGA*/*PGV* ratio and wide range of accelerations in their response spectra. They produce damaging and impulsive effects on structures, which require some special attentions. Most of the seismic design guidelines are developed based on the characteristics of far-field ground motions (Phan et al. 2007), although the near field earthquakes produce pulse-like motion and induce high input energy on structures. Demands made by near-fault motions have been shown to exceed the strength, displacement and ductility capacity of structures resulting in a large increase in the inter-storey drift, base shear demand for both long and short period structures (Alavi and Krawinkler 2001; Hall et al. 1995; MacRae et al. 2001). Liao et al. (2000) studied the dynamic behaviour of a five-span concrete pier bridge subjected to both near fault and far-field ground motions. The results show that the near-fault earthquake ground motions cause more ductility demands and base shear than far-field earthquake ground motions.

5.3 FRAGILITY FUNCTION METHODOLOGY

Fragility curves are usually two types, namely empirical and analytical. The empirical fragility curves are developed based on post earthquake surveys, which help provide a general idea about the relationship between the various damage states of the structures and the ground motion indices. However, limited damage data and subjectivity in defining damage states make this approach unrealistic for developing fragility curves for retrofitted bridges (Padget and

DesRoches, 2008). On the contrast, many analytical methods have been utilized to derive analytical fragility functions for expressing seismic vulnerability of a bridge, which include elastic spectral analyses (Hwang et al., 2000), nonlinear static analyses (Mander and Basoz, 1999; Shinozuka et al., 2000a), and linear/nonlinear time-history analyses (Hwang et al., 2001; Shinozuka et al., 2000b; Choi et al., 2004; Mackie and Stojadinovic, 2004). In this study probabilistic seismic demand model (*PSDM*) was used to derive the analytical fragility curves using nonlinear time-history analyses of the bridge system.

The *PSDM* establishes a correlation between the engineering demand parameters (*EDP*) and the ground intensity measures (*IM*). In the current study, the displacement ductility of bridge pier was considered as the *EDP*, and the peak ground acceleration (*PGA*) was utilized as intensity measure (*IM*) of each ground motion record.

Two approaches are used to develop the *PSDM*: the scaling approach (Zhang and Huo, 2009) and the cloud approach (Choi et al., 2004; Mackie and Stojadinovic, 2004; Nielson and Desroches, 2007a, b). In the scaling approach, all the ground motions are scaled to selective intensity levels and an incremental dynamic analysis (*IDA*) is conducted at each level of intensity; however, in the cloud approach, un-scaled earthquake ground motions are used in the nonlinear time-history analysis and then a probabilistic seismic demand model is developed based on the nonlinear time history analyses results. In the current study, only the cloud method was utilized in evaluating the seismic fragility functions of the retrofitted bridge bent. In the cloud approach, a regression analysis is carried out to obtain the mean and standard deviation for each limit state by assuming the power law function (Cornell et al. 2002), which gives a logarithmic correlation between median *EDP* and selected *IM*:

$$EDP = a\,(IM)^b \;\; or, \;\; ln\,(EDP) = ln\,(a) + b\,ln\,(IM) \qquad [5.1]$$

where, *a* and *b* are unknown coefficients which can be estimated from a regression analysis of the response data collected from the nonlinear time history analyses.

In order to create sufficient data for the cloud approach incremental dynamic analysis (*IDA*) is carried out instead of the nonlinear time history analyses. For carrying out the *IDA* the ground motions are not scaled to a particular intensity rather they are scaled from a very low *PGA* to the maximum *PGA* of the respective ground motion. The dispersion of the demand, $\beta_{EDP|\,IM}$, conditioned upon the *IM* can be estimated from Equation 5.2 (Baker and Cornell, 2006).

$$\beta_{EDP|\,IM} = \sqrt{\frac{\sum_{i=1}^{N}(ln(EDP)-ln\,(aIM^b))^2}{N-2}} \qquad [5.2]$$

With the probabilistic seismic demand models and the limit states corresponding to various damage states, it is now possible to generate the fragilities (the conditional probability of reaching a certain damage state for a given *IM*) using Equation 5.3 (Nielson, 2005).

$$P[LS|IM] = \varphi[\frac{ln(IM)-ln(IM_n)}{\beta_{comp}}] \qquad [5.3]$$

Where, $ln(IM_n) = \frac{ln(S_c)-ln\,(a)}{b}$ [5.4] *ln(IM_n)* is defined as the median value of the intensity measure for the chosen damage state (slight, moderate, extensive, collapse), *a* and *b* are the regression coefficients of the *PSDM*s and the dispersion component is presented in Equation 5.5 (Nielson, 2005).

$$\beta_{comp} = \frac{\sqrt{\beta_{EDP|IM} + \beta_c^2}}{b} \qquad [5.5]$$

where S_c is the median and β_c is the dispersion value for the damage states of the bridge pier. Table 5.1shows the damage states of bridge pier used in this study. Dutta and Mander (1999) recommended some limit states for bridge pier. But retrofit affects the seismic response and demand of the bridge bent and the capacity as well. For the retrofitted bridge pier new limit states need to be defined. Limit states capacities for all the four different retrofitted bridge bent are obtained by transforming the drift limits proposed by Dutta and Mander (1999) to ductility demand of the bridge pier. The limit state capacities, for the retrofitted bridge bents, presented in terms of median and dispersion, derived for this study is shown in Table 5.1.

Table 5.1: Limit states for retrofitted bridge pier

Retrofit technique	**Slight**		**Moderate**		**Extensive**		**Collapse**	
	S_c	β_c	S_c	β_c	S_c	β_c	S_c	β_c
CFRP	1.62	0.59	3.47	0.51	5.78	0.64	11.56	0.65
Concrete	1.81	0.59	3.89	0.51	6.48	0.64	12.96	0.65
Steel	2.25	0.59	4.82	0.51	8.04	0.64	16.08	0.65
ECC	1.72	0.59	3.70	0.51	6.16	0.64	12.32	0.65

5.4 SELECTION OF GROUND MOTION

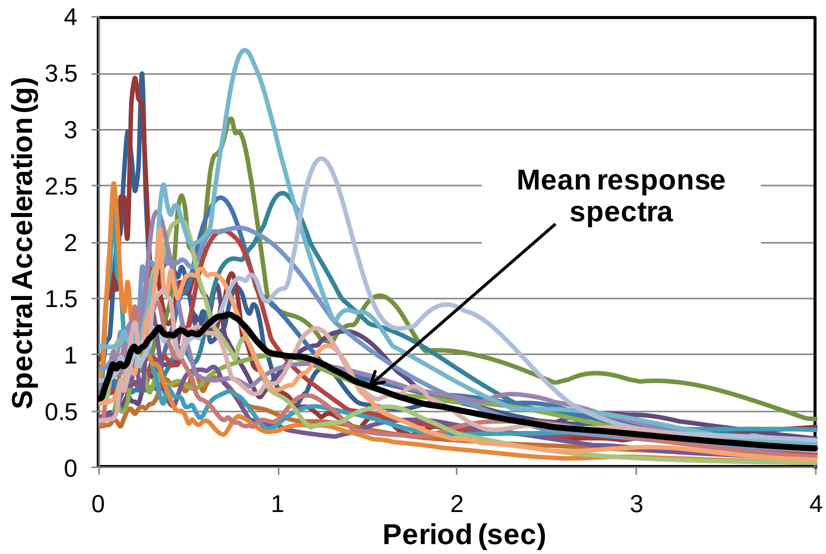
In order to establish a relationship between earthquake ground motion and structural damage, data set comprising of inputs (ground motion records) and outputs (damage) is necessary. To develop this fragility relationship, the methods usually employed include: a) collect the actual earthquake records and damage data, b) perform earthquake response analysis for given inputs and models, and c) subsequently obtain the resultant damages. But lack of adequate earthquake records and structural damage data makes the former one very difficult to implement although it is more realistic and convincing. On the contrary, the analytical method does not rely on actual seismic damage records and that make the process relatively easy to construct well-distributed data comprising input ground motions and structural damage. Nevertheless, uncertainty arising from a number of sources is present in the modeling and performance assessment of retrofitted bridges, which require careful consideration while selecting models of the structure and input ground motions. The nonlinear time history analyses take the nonlinearity of the members into account, and responses of the bridge are subsequently dependent on the characteristics of earthquake ground motions. So, the uncertainty characteristics of the earthquake ground motions regarding ground type, intensity and frequency contents have a great effect on nonlinear time history responses of members. Moreover, it is important to properly select input motion parameters to correlate with structural damage.

Selection of proper Intensity Measure (*IM*) plays a vital role in establishing fragility relationship. Mackie and Stojadinovic (2007) and Padgett et al., (2008) suggested that the peak ground acceleration (*PGA*) is the optimum index to describe the severity of the earthquake ground motion. However, as a large value of *PGA* is not always followed by severe structural damage, other intensity measures such as peak ground velocity (*PGV*) (Nielson, 2005), peak ground displacement (*PGD*), time duration of strong motion (T_d), spectrum intensity (*SI*) and spectral characteristics can also be considered. In this study *PGA* is used as the *IM* because of its efficacy, utility and adequacy in vulnerability assessment.

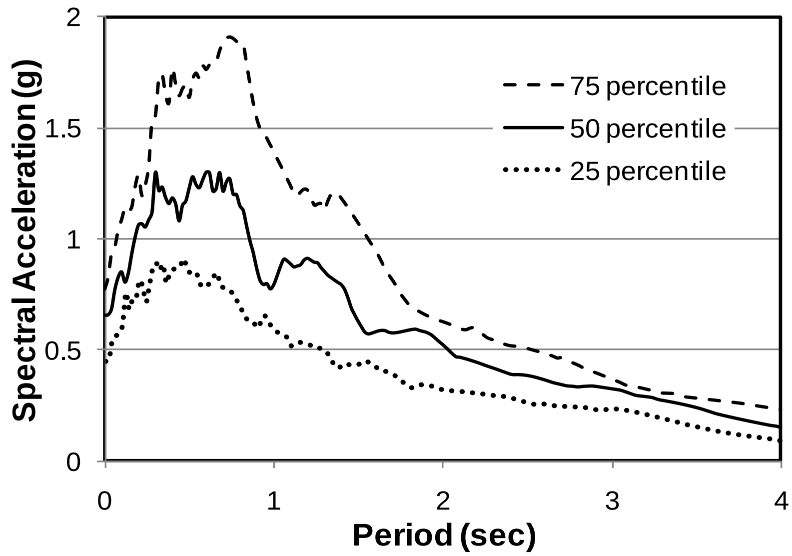
A suite of 20 near field and 20 far field ground motions are used in this study to develop fragility curves for the retrofitted bridge bent. The near field ground motions are obtained from SAC Joint Venture Steel Project Phase 2 (SAC, 2000). The characteristics of the earthquake ground motion records are presented in Table 5.2. All these ground motions have very high *PGA* ranging from 0.46g to 1.07g with epicentral distances less than 10 km.

Table 5.2: Characteristics of the near field ground motion histories

EQ No	Earthquake			Epicentral Distance (km)	PGA_{max} (g)	PGV_{max} (cm/s.)
	M	Year	Name			
1	7.4	1978	Tabas	1.2	0.9	108
2	7.4	1978	Tabas	1.2	0.958	103.8
3	7	1989	Loma Prieta	3.5	0.703	170
4	7	1989	Loma Prieta	3.5	0.458	89.33
5	7	1989	Loma Prieta	6.3	0.672	175
6	7	1989	Loma Prieta	6.3	0.37	67.34
7	7.1	1992	C. Mendocino	8.5	0.625	123.4
8	7.1	1992	C. Mendocino	8.5	0.65	91
9	6.7	1992	Erzincan	2	0.423	117
10	6.7	1992	Erzincan	2	0.448	57
11	7.3	1992	Landers	1.1	0.69	133.4
12	7.3	1992	Landers	1.1	0.79	69
13	6.7	1994	Nothridge	7.5	0.87	171
14	6.7	1994	Nothridge	7.5	0.381	59.7
15	6.7	1994	Nothridge	6.4	0.72	120
16	6.7	1994	Nothridge	6.4	0.583	52.9
17	6.9	1995	Kobe	3.4	1.07	157
18	6.9	1995	Kobe	3.4	0.563	71
19	6.9	1995	Kobe	4.3	0.77	170.5
20	6.9	1995	Kobe	4.3	0.424	62.5



(a)



(b)

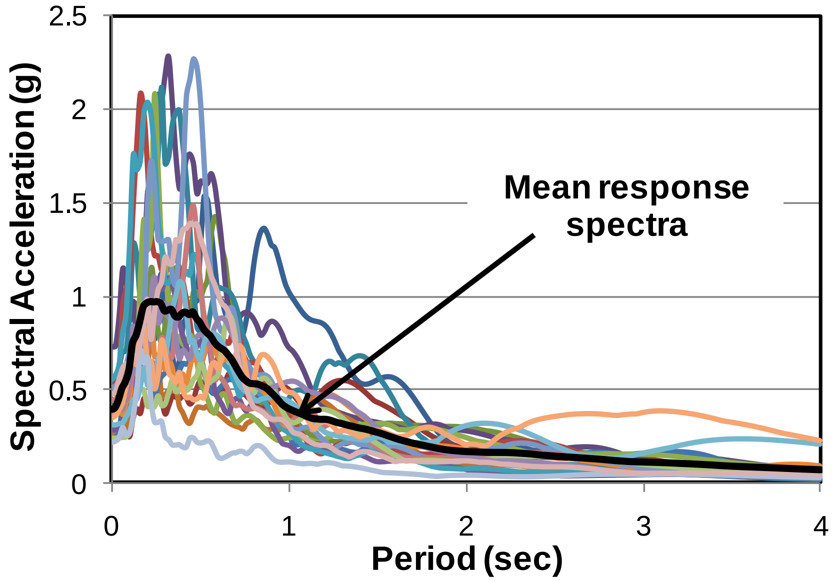
Figure 5.1: Earthquake ground motion records, (a) spectral acceleration, (b) percentiles of spectral acceleration of a suit of 20 near fault earthquake ground motion records

Figure 5.1a shows the acceleration response spectra with 5 percent damping ratio of the recorded near fault ground motions. The mean amplitude of the earthquake records is also presented in the figure. Figure 5.1b shows the different percentiles of acceleration response spectra with 5% damping ratio illustrating that the selected earthquake ground motion records are well describing the medium to strong intensity earthquake motion histories.

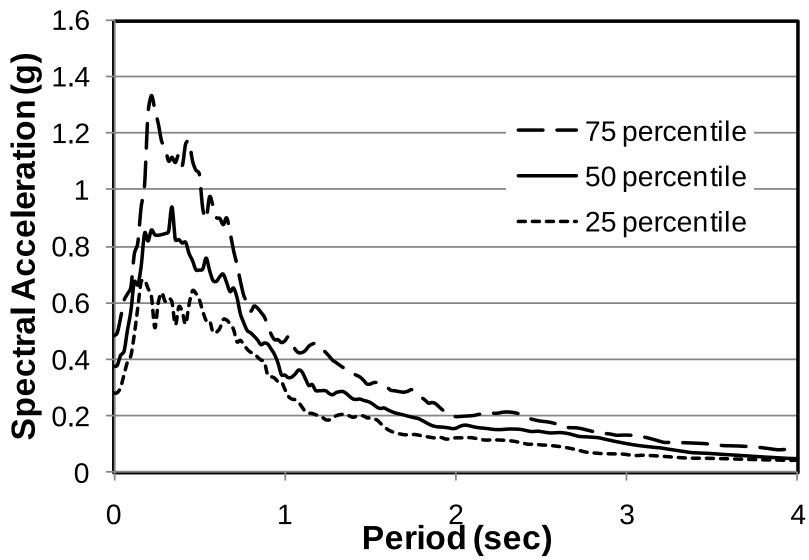
The 20 far field ground motion histories used in the study were obtained from the FEMA P695 (ATC-63) far-field ground motion set. The characteristics of the earthquake ground motion records are presented in Table 5.3 and Figure 5.2. All these ground motions have low to medium *PGA* ranging from 0.24g to 0.73g with epicentral distances more than 10km.

Table 5.3: Characteristics of the far field ground motion histories

EQ No	**Earthquake**			**Epicentral Distance (km)**	**PGA_{max} (g)**	**PGV_{max} (cm/s.)**
	M	**Year**	**Name**			
1	6.7	1994	Northridge	13.3	0.42	58.95
2	7.3	1992	Landers	86	0.24	52
3	6.7	1994	Northridge	26.5	0.41	42.97
4	7.3	1992	Landers	82.1	0.28	26
5	7.1	1999	Duzce, Turkey	41.3	0.73	56.44
6	6.9	1989	Loma Prieta	9.8	0.53	35
7	7.1	1999	Hector Mine	26.5	0.27	28.56
8	6.9	1989	Loma Prieta	31.4	0.56	36
9	6.5	1979	Imperial Valley	33.7	0.24	26
10	7.4	1990	Manjil, Iran	40.4	0.51	43
11	6.5	1979	Imperial Valley	29.4	0.36	34.44
12	6.5	1987	Superstition Hills	35.8	0.36	46
13	6.9	1995	Kobe, Japan	8.7	0.51	37.28
14	6.5	1987	Superstition Hills	11.2	0.45	36
15	6.9	1995	Kobe, Japan	46	0.24	38
16	7.0	1992	Cape Mendocino	22.7	0.39	44
17	7.5	1999	Kocaeli, Turkey	98.2	0.31	59
18	7.6	1999	Chi-Chi, Taiwan	32	0.35	71
19	7.5	1999	Kocaeli, Turkey	53.7	0.22	17.69
20	7.6	1999	Chi-Chi, Taiwan	77.5	0.47	37



(a)



(b)

Figure 5.2: Earthquake ground motion records, (a) spectral acceleration, (b) percentiles of spectral acceleration of a suit of 20 far field earthquake ground motion records

5.5 CHARACTERIZATION OF DAMAGE STATES

The probability of entering a damage state given an input ground motion intensity parameter is expressed by fragility curves. Damage states for bridges should be defined in such a way that each damage state indicates a particular level of bridge functionality. Different forms of *EDPs* are used to measure the *DS* of the bridge components. Based on energy dissipation capacity and ductility demand of structure, Park and Ang (1985) developed a damage index while Hwang et al. (2000) used the capacity/demand ratio of the bridge columns as *EDP* to develop fragility curves. A capacity model is needed to measure the damage of bridge component based on prescriptive and descriptive damage states in terms of *EDP*s (FEMA, 2003, Choi et al., 2004, Nielson, 2005).

Four damage states as defined by HAZUS (FEMA, 2003) are commonly adopted in the seismic vulnerability assessment of engineering structures, namely slight, moderate, extensive and collapse damages. The details of the different damage states adopted in this study was discussed in detail in section 4.5.2.3 and Table 4.5 summarized the definitions of various damage states and their corresponding damage criteria. Bridge piers are one of the most critical components, which are often forced to enter into nonlinear range of deformations under strong earthquakes. In this study, the displacement ductility of the bridge pier is adopted as *DI*, and the corresponding limiting values are those shown in Table 4.5.

5.6 BRIDGE BENT DETAILS AND ANALYTICAL MODELING

The details of the bridge bent have been described in Section 4.2. Detailed description and mechanical properties of the retrofitting materials are presented in section 4.2.1 and Table 4.1.

The details about the analytical model for these retrofitted bridge bents have been described in Section 4.2.2.

5.7 FRAGILITY ANALYSIS OF RETROFITTED BRIDGE BENT

In this study probabilistic seismic demand models are used to derive the fragility curves. The *PSDM*s help to express the effect of a given retrofit technique on the seismic demand placed on the retrofitted bridge bent column. The demand parameter considered in this study is the column displacement ductility demand. The findings are unique for each retrofit measure considered. The *PSDM*s are developed by analyzing the demand placed on the retrofitted bridge bent through a regression analysis. *PSDM*s are constructed from the peak response of the bent column obtained from the *IDA.* Figure 5.3 shows the *PSDM*s for retrofitted bridge bent for near field ground motions. For generating the *PSDM*s a suite of suitable ground motions representing a broad range of values for the selected *IM* (*PGA* in this study) was chosen. After the development of analytical models of retrofitted bridge bents, *IDA* was carried out. From each analysis the peak responses were calculated and plotted against the *IM* for that ground motion. Finally a regression analysis was carried out to estimate a, b and $\beta_{EDP|IM}$.

The impact of four different retrofit measures on the demand models is compared in Table 5.4. The parameters listed represent the regression parameters from Equation 5.1 along with the dispersion. From the table it is evident that the steel jacketing yields an increase in the dispersion in the demand ($\beta_{D|IM}$) while ECC jacketing exhibited reduction in the dispersion in the demand. On the other hand the concrete jacketing increases the median value of the demands placed on the columns, exhibited by an increase in the parameters affecting both the intercept ($ln(a)$) and slope (b) of the regression model. Different retrofits have different relative effects on the column ductility demand. It revealed that ECC jacket was the most effective in reducing the demand followed by CFRP, concrete and steel jacketing.

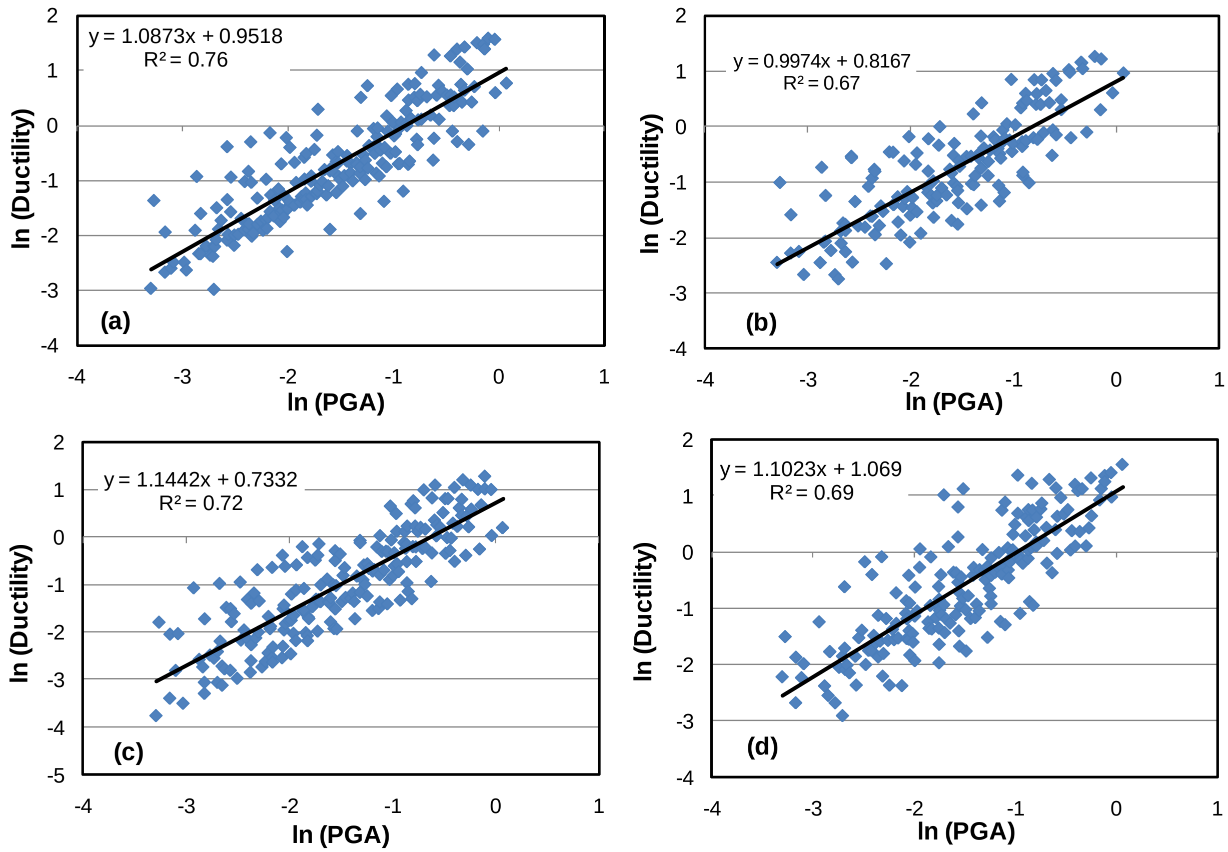


Figure 5.3: Comparison of the *PSDM*s for bridge bent retrofitted with (a) steel jacketing, (b) CFRP jacketing, (c) ECC jacketing and (d) concrete jacketing for near field ground motion

Table 5.4: *PSDM*s for four different retrofits of the bridge bent (near field)

	Column Ductility		
Retrofit Technique	***ln (a)***	***b***	$\beta_{EDP\|IM}$
Steel Jacketing	0.9518	1.0873	0.56
CFRP Jacketing	0.8167	0.9974	0.51
ECC Jacketing	0.7332	1.1442	0.48
Concrete Jacketing	1.069	1.1023	0.54

Evaluation of the fragility curves offers a valuable insight on the effectiveness of various retrofit measures on the probability of the damage considering both the impact of retrofit on the bridge's demand and capacity. Figure 5.4 presents the fragility curves of the retrofitted bridge bent with four different techniques under near fault ground motions. The fragility can be directly estimated from the limit state capacity of each damage state (Table 5.1) as well as the parameters for the *PSDM*s obtained from regression analysis. Utilizing these parameters, the fragility curves were generated using equation 5.3. The figures facilitate the comparison of the relative effectiveness of different retrofit measures for the selected bridge bent. These fragility curves do provide information about the most vulnerable bridge bent. These plots of various damage state aid in expressing the effect of a retrofit measure that can vary dramatically from one damage state to another. Evaluation of the fragilities (shown in Figure 5.4) for the retrofitted bridge bent under near fault ground motions indicate that for all the damage states from *slight* to *collapse*, the concrete jacketing retrofit is the most vulnerable measure that has a considerable impact on the fragility of the bridge bent. On the other hand the ECC and CFRP jacket essentially eliminated the vulnerability of the bridge bent, yet ECC jacket had higher probability of collapse as compared to CFRP jacketed bridge bent.

Table 5.5 summarizes the mean and standard deviation of intensity measure required to reach four damage states obtained by the *PSDM* method. Assuming a lognormal distribution, two lognormal distribution parameters λ and ξ^2 for the bridge bent, the mean and standard deviation of the logarithmic *IM*s, are evaluated using a standard regression analysis method. The values of the two parameters are presented in Table 5.5.

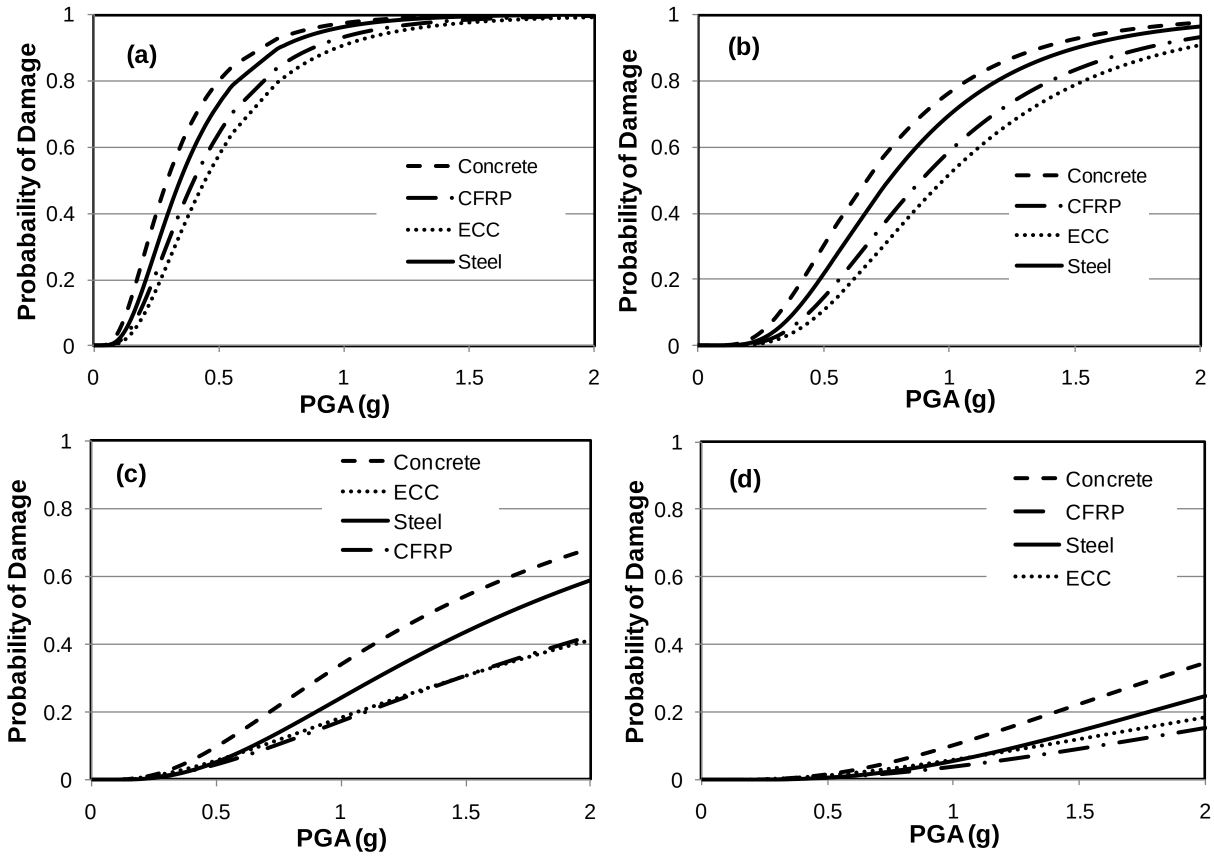


Figure 5.4: Fragility curves for the retrofitted bridge bent for: (a) slight damage, (b) moderate damage, (c) extensive damage, and (d) collapse, under near fault ground motion

Table 5.5: Parameters of fragility curves for the bridge bent with respect to *PGA* (near field)

Damage state	**Slight**		**Moderate**		**Extensive**		**Collapse**	
Type of Retrofit	λ	ξ^2	λ	ξ^2	λ	ξ^2	λ	ξ^2
Steel Jacketing	-0.64	0.84	-0.19	0.79	0.28	0.86	0.65	0.87
CFRP Jacketing	-0.56	0.93	-0.07	0.88	0.44	0.96	0.84	0.97
ECC Jacketing	-0.42	0.96	0.01	0.88	0.46	1.01	0.81	1.02
Concrete Jacketing	-0.74	0.86	-0.29	0.81	0.17	0.88	0.53	0.89

Figure 5.5 shows the *PSDM*s for retrofitted bridge bent for far field ground motions. The impact of the four different retrofit measures under far field ground motions on the demand models is compared in Table 5.6. The two lognormal distribution parameters λ and ξ^2 for the retrofitted bridge bents under near field ground motion are summarized in Table 5.7.

Table 5.6: *PSDM*s for four different retrofits of the bridge bent (far field)

	Column Ductility		
Retrofit Technique	***ln (a)***	***b***	$\boldsymbol{\beta_{EDP\mid IM}}$
Steel Jacketing	0.9178	1.1041	0.39
CFRP Jacketing	0.8135	1.085	0.38
ECC Jacketing	0.7578	1.0713	0.39
Concrete Jacketing	0.9845	1.1023	0.38

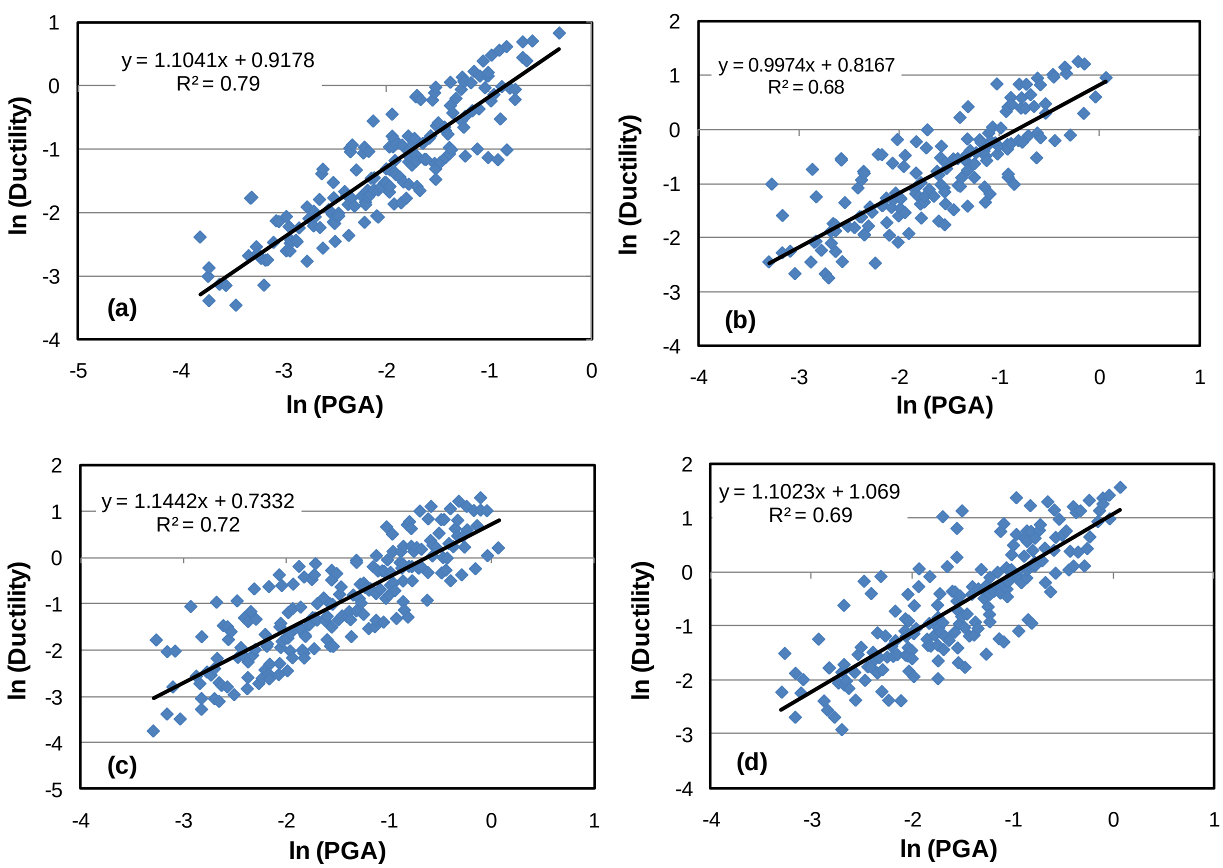


Figure 5.5: Comparison of the *PSDM*s for bridge bent retrofitted with (a) steel jacketing, (b) CFRP jacketing, (c) ECC jacketing and (d) concrete jacketing for far field ground motion

Table 5.7: Parameters of fragility curves for the bridge bent with respect to *PGA* (far field)

Damage state	**Slight**		**Moderate**		**Extensive**		**Collapse**	
Type of Retrofit	λ	ξ^2	λ	ξ^2	λ	ξ^2	λ	ξ^2
Steel Jacketing	-0.59	0.78	-0.16	0.73	0.31	0.80	0.67	0.82
CFRP Jacketing	-0.51	0.78	-0.07	0.74	0.41	0.81	0.77	0.82
ECC Jacketing	-0.46	0.78	-0.01	0.73	0.46	0.81	0.82	0.82
Concrete Jacketing	-0.68	0.79	-0.23	0.75	0.25	0.83	0.63	0.84

Plots of the fragility curves for the bridge bents retrofitted with four different options are shown in Figure 5.6, which illustrate the relative vulnerability of the retrofitted bridge bents over a range of far field earthquake intensities and damage states. From Figure 5.6 it is evident that the different retrofit measures appear to be more effective for different damage states in terms of reducing the probability of the damage for a given *PGA*. For example, the steel jacketing reduces the vulnerability for the bridge bent all-through from slight to collapse damage state as compared to the concrete jacketing. From each figure it is revealed that the ECC jacketing in the bridge bent considerably reduced the vulnerability at all four damage states. This is a result of the influence of the retrofit measures on the seismic capacity and demand placed on the bridge bent. The relative vulnerability of various retrofitting options varies considerably depending on the damage state.

Finally, the median of the probability of exceedence is determined for the bridge bent retrofitted with different retrofitting techniques at each damage level. Figure 5.7a shows a plot of the peak ground accelerations for the median values of probability of damage of the bridge bent retrofitted with different retrofitting techniques for near field ground motions. From the figure it is revealed that the bridge bent retrofitted with concrete jacketing portrays seismically more fragile than that of other techniques at each damage state. For the slight damage state, the median *PGA* for the four retrofit types range from 0.48 g for the concrete jacket to 0.66 g for the ECC jacketed bridge bent. For the complete damage state, the median *PGA* ranges from approximately1.71 g for the concrete jacket to 2.32 g for the CFRP jacketed bridge bent.

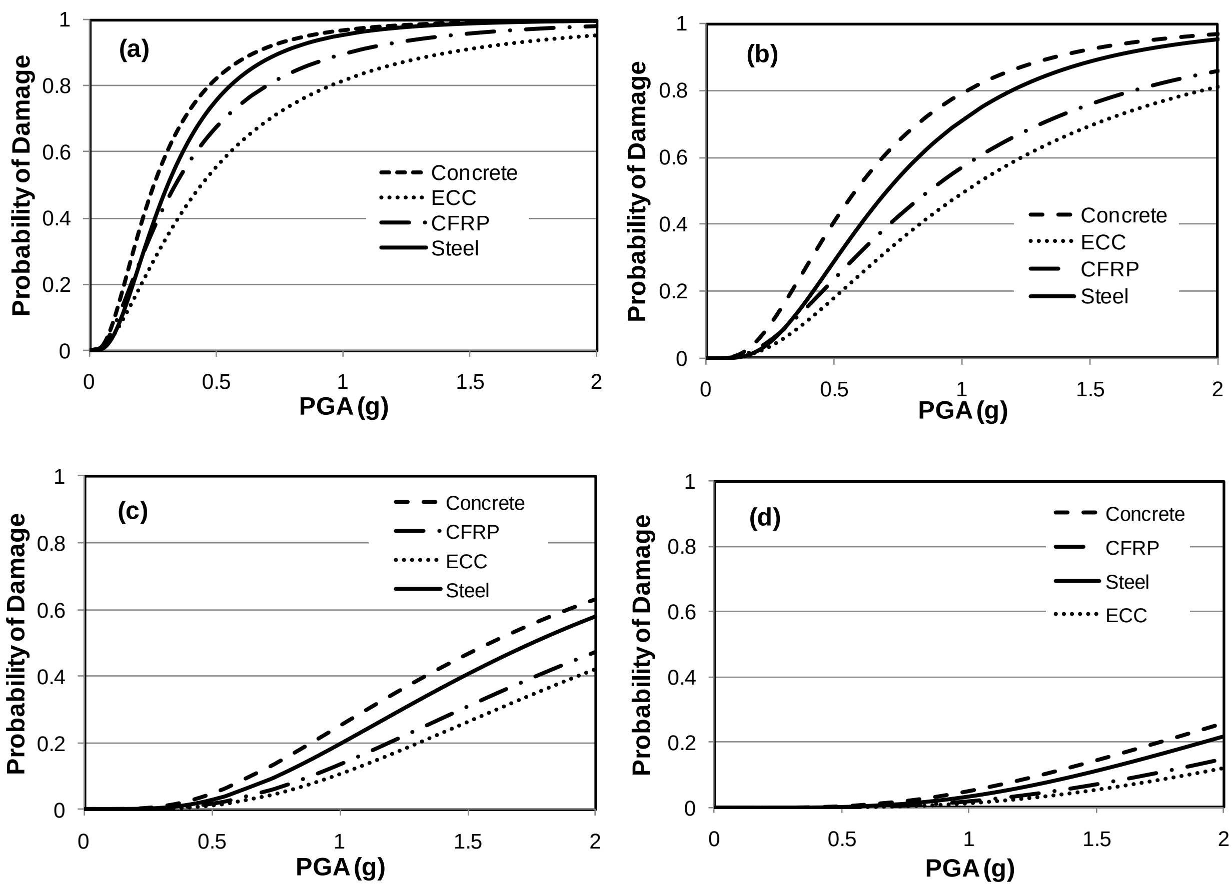
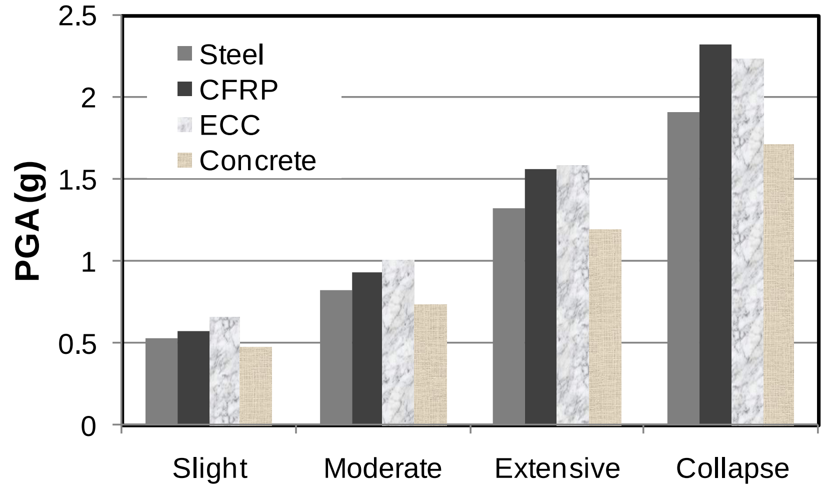
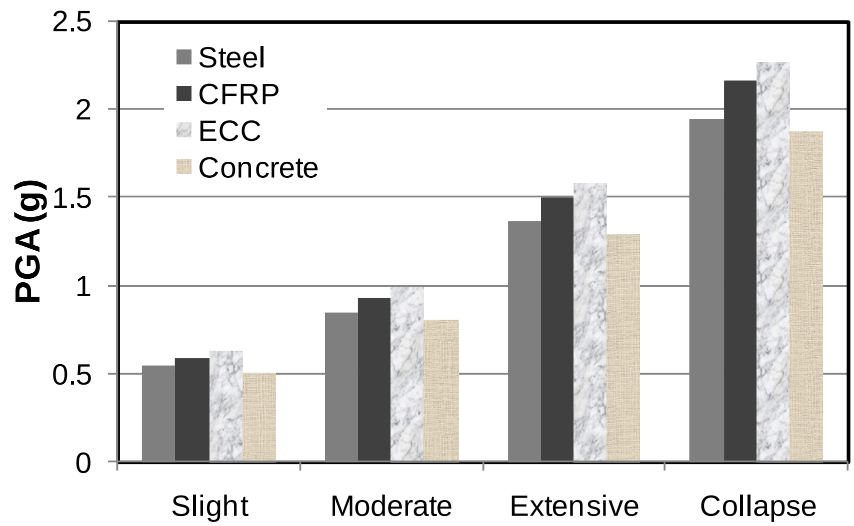


Figure 5.6: Fragility curves for the retrofitted bridge bent for: (a) slight damage, (b) moderate damage, (c) extensive damage, and (d) collapse, for far field ground motion

The results of this analysis confirm previous results that have shown that the concrete jacketed bent is the most vulnerable of the four retrofit techniques used in this study. The similar trend is also observed in the retrofitted bridge bents for far field ground motions as presented in Figure 5.7b, which indicate that the bridge bent with concrete jacketing is seismically more vulnerable than the other three techniques adopted in this study.



(a)



(b)

Figure 5.7: Comparison of median values of *PGA* for the bridge bent retrofitted with different retrofitting techniques for (a) near field ground motion and (b) far field ground motion

5.8 SUMMARY

This study adopted the performance-based evaluation approach to investigate the effectiveness of different retrofitting methods so as to minimize the overall damaging potential of seismically vulnerable bridge bents. To investigate the seismic vulnerability of the retrofitted bridge bents, a total of 40 earthquake excitations of which 20 are near field and 20 are far field ground motions, are utilized to evaluate the likelihood of exceeding the seismic capacity of the retrofitted bridge bents. The use of fragility curves for retrofitted bridge bents aided in expressing the potential impact of retrofit on the bridge bent vulnerability. The results obtained from this study indicates that the ECC jacketed bridge bent possess less vulnerability at all damage state under both near and far field earthquakes.

CHAPTER 6 : PERFORMANCE BASED MULTI-CRITERIA DECISION MAKING FOR SEISMIC RETROFITTING OF BRIDGE BENT

6.1 GENERAL

Highway bridges are expected to perform their intended function continuously while maintaining a level of safety and serviceability throughout their planned life. These important highway structures very often encounter catastrophic seismic events, which eventually result in collapse and severe monetary losses. Therefore, those structures lacking sufficient resistance to earthquake need to be retrofitted. But engineers, decision makers and highway agencies are often faced with the dilemma whether or not to retrofit existing bridges and lower their potential losses due to seismic events. In case of the critical facilities, such as highway bridges, the decision on the most suitable retrofit solution may provide a consistent basis for intervention design and construction management. But this decision is associated with a variety of factors which can affect the consequences of a decision (Hall and Wiggins 2000). Selection of a suitable retrofit alternative involves (a) a group of potential alternatives, (b) multiplicity of criteria to distinguish among the objectives (c) experts from different sphere and (d) consequences of decisions, are just a few sources leading the optimal selections to a gray area. In such multi-discipline engineering paradigms, multi-criteria decision making (MCDM) methods can provide a feasible solution for the implementation of retrofit strategies for seismic risk mitigation.

Performance evaluation and optimal selection of retrofit techniques have multi-level and multi-factor features and therefore, be regarded as multiple criteria decision-making problem. This study discusses the application of an MCDM method, known as TOPSIS (Technique for

Order Preference by Similarity to Ideal Solution by Hwang and Yoon, 1981) for the selection of the optimal retrofit strategy in the case of an under-designed (pre-1965) RC multi column bridge bent. The selected MCDM method is capable of performing the solution procedure regardless of the functional relationship for the objectives and constraints, and can handle a number of alternatives and criteria (Milani et al. 2005). Moreover, the computation processes are straightforward and the method permits the pursuit of best alternatives for each criterion depicted in a simple mathematical form (Wang and Chang, 2007).

The structure considered in this study is the northbound lanes of the South Temple Bridge, which was built in the year 1963 and had several deficiencies in the amount and seismic detailing of the steel reinforcement. The author aimed at enhancing the seismic capacity (enhancing both ductility and strength) of the bridge bent using different retrofitting schemes for instance, providing confinement with Carbon Fiber Reinforced Plastics (CFRP); Engineered Cementitious Composites (ECC), concrete jacketing and steel jacketing.

The main objective of this research is to propose a simplified and systematic approach for selecting a suitable seismic retrofit technique among a set of available alternatives. Retrofit selection is both an MCDM problem where many conflicting criteria should be considered in decision making, and a problem containing subjectivity, ambiguity and uncertainty in the assessment process. Therefore, this research uses entropy method (Hwang and Yoon, 1981) to determine the weighting of a given set of criteria and TOPSIS to determine performance ratings of the feasible alternatives. The decision process is made of the four steps and the entire process is depicted in Figure 6.1.

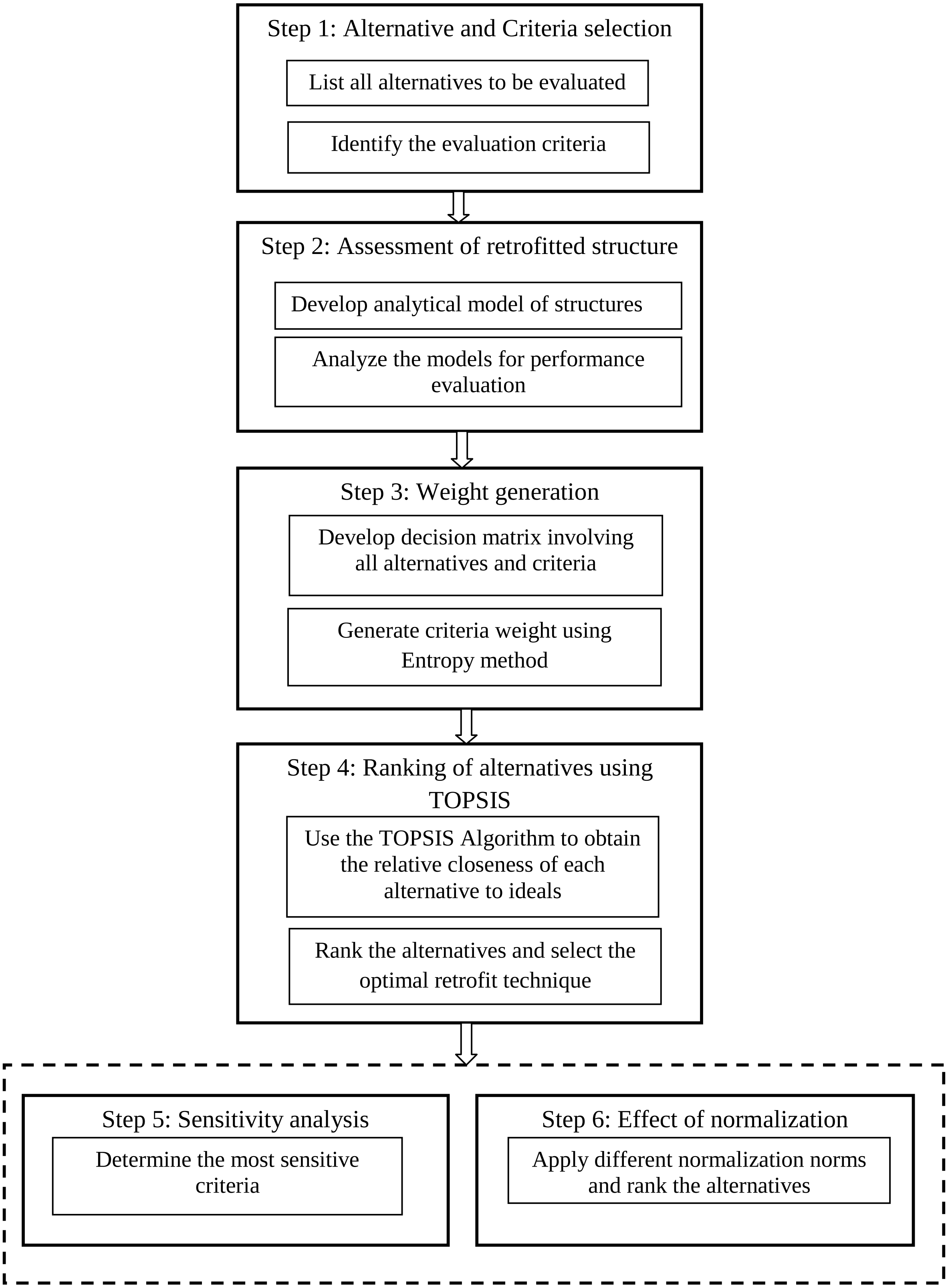


Figure 6.1: Information flow for ranking retrofit alternatives

6.2 TOPSIS

TOPSIS (Technique for order preference by similarity to ideal solution) is a powerful tool for handling ranking multi-attribute/criteria decision making (MCDM) problems. Hwang and Yoon (1981) described the TOPSIS concept, with the reference to the positive and negative ideal solutions, as the ideal and anti-ideal solutions, respectively. The TOPSIS method defines an index called similarity (or relative closeness) to rank the alternatives based on the distance (or similarity) of their evaluated score from the ideal solution in a MCDM problem.

TOPSIS method has been extensively used and modified by many researchers to deal with MCDM problems. The reasoning for selecting this method can be seen in its high speed, accuracy, and compatibility. The method is based on the geometrical concept that the best alternative should have the shortest distance to a *positive ideal* solution (A^*) and the farthest distance to a *negative-ideal* one (A^-). TOPSIS assumes that each criterion wants to be either maximized or minimized, so the *positive ideal* solution for a criterion is the "max-value" of all the alternatives considered, and the *negative-ideal solution* is the "min-value" of the criterion for all alternatives. A five step algorithm of TOPSIS is discussed further below following Hwang and Yoon (1981).

6.2.1 Step 1: Construct the Normalized Decision Matrix

In the first step, a normalized decision (or evaluation) matrix is constructed. A decision matrix $\boldsymbol{D}$ is an (m x n) matrix in which element x_{ij} indicates the performance of alternative A_i when it is evaluated in terms of decision criterion C_j (for i= 1, 2,3,......,m and j= 1,2,3,.....,n), which results in the following evaluation matrix.

$$\boldsymbol{D} = \begin{array}{c} \\ A_1 \\ A_2 \\ - \\ - \\ - \\ A_i \end{array} \begin{array}{c} \begin{array}{cccccc} C_1 & C_2 & - & - & - & C_j \end{array} \\ \begin{bmatrix} x_{11} & x_{12} & - & - & - & x_{1j} \\ x_{21} & x_{22} & - & - & - & x_{2j} \\ - & - & - & - & - & - \\ - & - & - & - & - & - \\ - & - & - & - & - & - \\ x_{i1} & x_{i2} & - & - & - & x_{ij} \end{bmatrix} \end{array} \qquad [6.1]$$

All the x_{ij} have to be collected in the decision matrix $\boldsymbol{D}$ = [x_{ij}] representing the starting point for any application of the TOPSIS method. In this case the data for decision matrix will be obtained from the analysis of the bridge bents. The normalization of x_{ij} values, each of those being characterized by different units, has to be done. According to the TOPSIS procedure, Equation (6.2) is adopted to normalize the decision matrix. Let r_{ij} indicate the normalized value of x_{ij}. The normalized decision matrix $\boldsymbol{R}$ = [r_{ij}] is thus obtained.

$$r_{ij} = \frac{x_{ij}}{\sqrt{\sum_{i=1}^{m} x_{ij}^2}} \quad , \qquad [6.2] \quad i=1, 2,3,......, m \; ; \; j = 1, 2, 3,......,n$$

6.2.2 Step 2: Construct the Weighted Normalized Decision Matrix

A set of weights W= (w_1, w_2, ..., w_j) derived using Entropy method used (see section 6.3.2) in conjunction with the above mentioned normalized decision matrix to determine the weighted normalized decision matrix $\boldsymbol{V}$ = [v_{ij}].

The weighted normalized decision matrix $\boldsymbol{V}$ is obtained as $v_{ij} = w_j \, x \, r_{ij}$, where w_j is the entropy weight of each criteria.

6.2.3 Step 3: Determine the Positive-Ideal and the Negative-Ideal Solutions

The positive ideal solution (*PIS, A**) is a solution that maximizes the benefit criteria (*B*) and minimizes the cost criteria (*C*), whereas the negative ideal solution (*NIS,* A^-) maximizes the cost criteria and minimizes the benefits criteria (Dagdeviren et al. 2009). The (*PIS, A**) and (*NIS,* A^-) are determined as follows:

$A^* = \{v_1^*, v_2^*, v_3^*, \ldots\ldots\ldots, v_j^*\} = \{(\max c_{ij} \mid j \in B), (\min a_{ij} \mid j \in C)\}$ [6.3]

$A^- = \{v_1^-, v_2^-, v_3^-, \ldots\ldots\ldots, v_j^-\} = \{(\min c_{ij} \mid j \in B), (\max a_{ij} \mid j \in C)\}$ [6.4]

6.2.4 Step 4: Calculate Distances from the Ideal Solutions

The separation (distance) between alternatives can be measured by the n-dimensional Euclidean distance. The separation of each alternative from the *PIS, A**, is given by:

$$S_i^* = \sqrt{\sum_{j=1}^{n} (v_{ij} - v_j^*)^2}, \qquad [6.5]$$

Similarly, the separation from the *NIS,* A^-, is given by:

$$S_i^- = \sqrt{\sum_{j=1}^{n} (v_{ij} - v_j^-)^2}, \qquad [6.6]$$

6.2.5 Step 5: Calculate the Relative Closeness to the Ideal Solution

The TOPSIS method ranks alternative solutions in terms of the so-called *relative closeness* C_i^* to the *PIS, A**, with C_i^* is calculated using Equation (6.7)

$$C_i^* = \frac{S_i^-}{S_i^* + S_i^-} \qquad [6.7]$$

6.2.6 Step 6: Rank the Preference Order

The best satisfied alternative can now be decided according to preference rank order of C_i^*. It is the one which has the highest C_i^* value i.e. the shortest distance from the ideal solution.

6.3 PERFORMANCE BASED RETROFIT SELECTION USING TOPSIS

The aim of this chapter is to select a suitable seismic retrofit technique for seismic upgrading of a multi-column bridge bent among a set of available alternatives using TOPSIS. Selection of best retrofit technique consists of four steps. Step 1 entails the selection of alternatives and criteria. In Step 2, the retrofitted bridge bents are analyzed using nonlinear static pushover (*SPO*) and incremental dynamic analysis (*IDA*) to evaluate their performance. In Step 3, the weight of each criterion is determined using Entropy method. Phase 3 involves an identification of the most suitable retrofit technique through ranking based on evaluation using TOPSIS method. Figure 6.1 shows the sequence of information flow for TOPSIS based seismic retrofit selection model.

6.3.1 Step-1: Selection of Alternatives and Criteria

Seismic rehabilitation denotes an approach aiming at achieving satisfactory seismic performance of an existing structure under strong ground motion. In an attempt to reduce the seismic vulnerability of bridge columns, a number of column strengthening techniques, such as steel jacketing, use of composite material jackets, ferrocement jacketing and jacketing with additional reinforced concrete have been tested and widely used in several earthquake-prone countries (Andrews and Sharma 1988, Rodriquez and Park 1994, Saadatmanesh et al. 1994, Masukawa et al. 1997, Priestley et al. 1997, Lehman et al. 2001, Kumar et al. 2005). Therefore, several options are available for upgrading an existing bridge structure and the decision maker has to select the most suitable one. Furthermore, it is extremely difficult to define the best retrofit solution in absolute terms, and the selection process strongly depends on the case under consideration. In order to make such a choice, a set of feasible alternative interventions has to be defined. The choice of retrofitting alternatives should be selected based on the specific features of the structure and its seismic deficiencies obtained during the assessment. The retrofit alternatives considered should be designed in such a way that they are comparable to each other.

For the MCDM application included in the study, a group of four alternatives was considered, all of those aiming at seismic capacity enhancement of the non-seismically designed bridge bent. The alternatives considered here are ECC jacketing (A_1), Steel jacketing (A_2), CFRP jacketing (A_3) and Concrete jacketing (A_4). Details of these four retrofitting techniques have been presented in section 4.2.1.

The proper selection of criteria is an important step in MCDM. If the selection of attributes (criteria) and alternatives is not carefully made, the solving algorithm can yield fatal errors. During a decision path, engineers must select criteria with superior performance while maintaining minimal interactions between them (Chen and Lin 2002). Criteria can be generally defined as different points of view from which the same solution can be evaluated. Since the bridge need to be operational after an earthquake event with very little or no damage, the evaluation criteria are selected based on their performance during an earthquake event. The designer has to choose the best retrofit option from these four available options based on their performance during an earthquake event. The criteria for selecting best retrofit method may involve the relative performance, cost and some other non technical aspects. This study focuses only on the performance criteria as safety is the prime concern for vital facilities such as bridges. As different performance indices are considered for performance-based design and assessment of structures, it becomes a crucial problem for the engineers to select suitable retrofit strategy that meet a set of predefined limits corresponding to a desired performance level (*PL*). The performance criteria selected for this study were Base shear capacity demand ratio (C_1), Residual Displacement (C_2), Ductility Capacity (C_3) and Energy dissipation capacity (C_4). For all the criteria except the residual displacement (C_2), the higher the value, the better is the performance of the retrofit option.

After determining the alternative retrofitting techniques and the criteria to be used in evaluation, a decision hierarchy can be described simply by three levels (Figure 6.2). The first level describes the goal or focus of the decision problem, which is described as the "Selection of retrofit method". The different criteria are placed on the second level, and the last level denotes the alternatives.

6.3.2 Step-2: Assessment of Retrofitted Bridge Bent

The details of the bridge bent have been described in Section 4.2. The details about the analytical model for these retrofitted bridge bents have been described in Section 4.2.2. The details of nonlinear static pushover analysis and the results have been discussed in Section 4.4.

6.3.2.1 Incremental Dynamic Time History Analysis

Details of the incremental dynamic analysis have been provided in section 4.5. In this chapter 20 selected earthquake records have been used for incremental dynamic analyses as shown in Table 6.1. The acceleration response spectrum (5% damped) for the selected ground motion sets is shown in Figure 6.3. The dynamic analyses were carried out for all bridge bents. The *IDA* results are used to compute the base shear capacity demand ratio, residual displacement and energy dissipation capacity. The capacity of the structure was obtained from the static non linear pushover analyses and the demand of the structures obtained from the dynamic analyses.

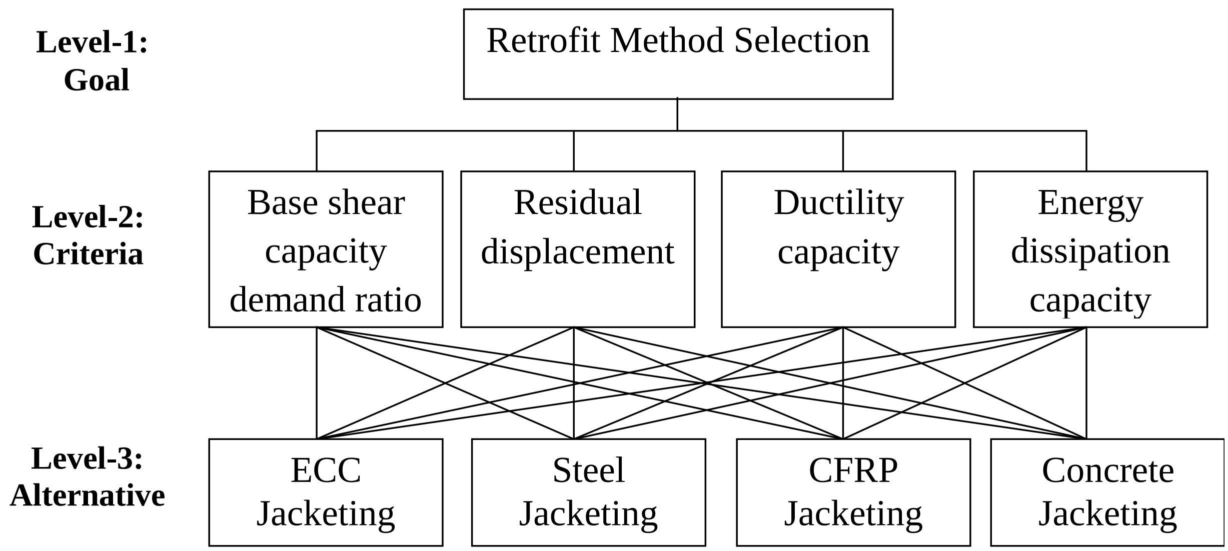


Figure 6.2: Hierarchy for the selection of seismic retrofit

Table 6.1: Selected earthquake ground motion records

No	Event	Year	Record Station	Φ^1	M^{*2}	R^{*3} (km)	PGA (g)
1	Imperial Valley	1979	Plaster City	45	6.5	31.7	0.042
2	Imperial Valley	1979	Plaster City	135	6.5	31.7	0.057
3	Imperial Valley	1979	Cucapah	0	6.9	16.9	0.309
4	Imperial Valley	1979	El Centro Array#13	140	6.5	21.9	0.117
5	Imperial Valley	1979	El Centro Array#13	230	6.5	21.9	0.139
6	Imperial Valley	1979	Westmoreland fire stn.	90	6.5	15.1	0.074
7	Imperial Valley	1979	Westmoreland fire stn.	180	6.5	15.1	0.11
8	Imperial Valley	1979	Chihuahua	282	6.5	28.7	0.254
9	Loma Prieta	1989	Coyote Lake Dam	285	6.5	22.3	0.179
10	Loma Prieta	1989	Agnews state hospital	90	6.9	28.2	0.159
11	Loma Prieta	1989	Sunnyvale Colton Ave	270	6.9	28.8	0.207
12	Loma Prieta	1989	WAHO	0	6.9	16.9	0.37
13	Loma Prieta	1989	WAHO	90	6.9	16.9	0.638
14	Loma Prieta	1989	Hollister Diff. Array	165	6.9	25.8	0.269
15	Loma Prieta	1989	Hollister Diff. Array	255	6.9	25.8	0.279
16	Loma Prieta	1989	Sunnyvale Colton Ave	360	6.9	28.8	0.209
17	Loma Prieta	1989	Holister south & Pine	0	6.9	28.8	0.371
18	Loma Prieta	1989	Anderson dam	270	6.9	21.4	0.244
19	Superstition Hill	1987	Wildlife liquefaction array	36	6.7	24.4	0.2
20	Superstition Hill	1987	Wildlife liquefaction array	90	6.7	24.4	0.18

[1]Component,
[2]Moment Magnitudes,
[3]Closest Distances to Fault Rupture
Source: PEER Strong Motion Database, http://peer.berkeley.edu/svbin

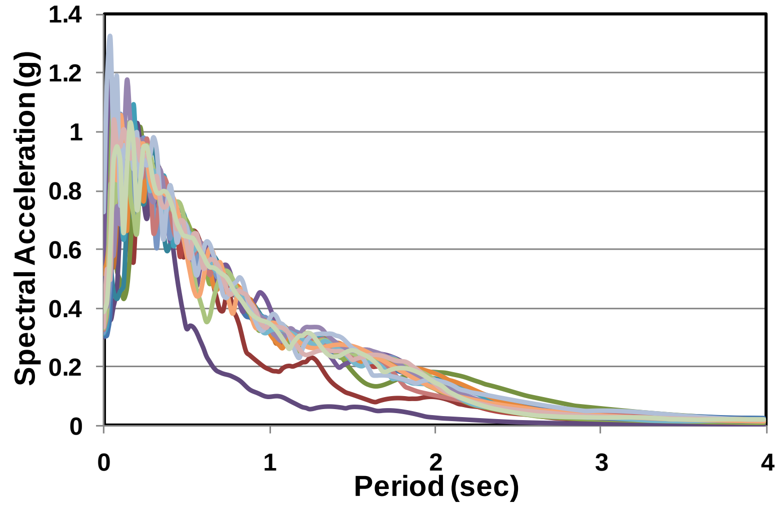
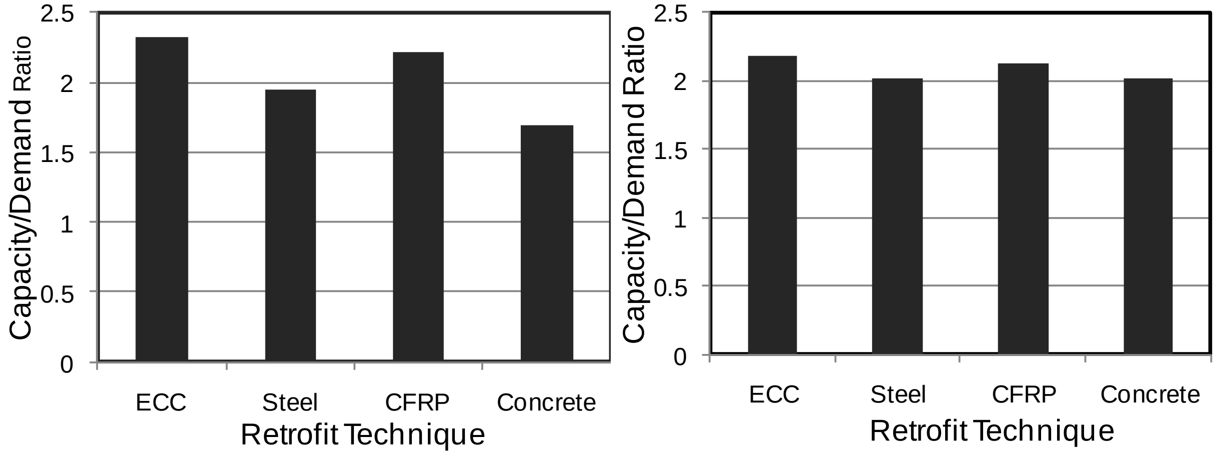


Figure 6.3: Spectral acceleration for selected ground motions

6.3.2.1.1 Base Shear Capacity/Demand Ratio

Figure 6.4 shows the base shear capacity demand ratio of four different retrofit techniques. Figure 6.4a depicts the base shear capacity demand ratio for ground motion-3, averaged over different intensity level. From this figure it is observed that the capacity demand ratio in terms of base shear for the retrofitted bridge bents varies with different retrofitting techniques. Figure 6.4b depicts the base shear capacity demand ratio averaged over 20 ground motions. From Figure 6.4 it is evident that bridge bent retrofitted with ECC jacketing has higher C/D ratio in terms of base shear. In Figure 6b it is seen that ECC jacketing showed a C/D ratio of 2.18, which is 2.8%, 8.5% and 7.9% higher than CFRP, Concrete and steel jacketed bridge bent, respectively. This average value for 20 ground motions has been used as the criteria value in the MCDM analysis.



(a) (b)

Figure 6.4: Base Shear C/D comparison of four retrofitting techniques (a) average of ground motion-3 at different intensity level (b) average of 20 ground motion

6.3.2.1.2 Residual Displacement

Residual displacement is an important parameter for performance based earthquake engineering. A comparison between the residual displacements of the different retrofitted bridge bents is shown in Figure 6.5. Figure 6.5a depicts the residual displacement for ground motion -3 averaged over different intensity level. There is a clear difference in the response of the four retrofitting systems. With increasing intensity all the retrofitted bridge bent begin to experience significant residual displacement with large variation in magnitudes. The bridge bent retrofitted with ECC jacketing experienced lowest residual displacement with a maximum of only 52 mm as shown in Figure 6.5a. On the other hand, the bridge bents retrofitted with steel and concrete jacketing retained a maximum residual drift of 68 mm and 75 mm, respectively. Bridge bent retrofitted with CFRP jacketing experienced substantially lower residual displacement (55 mm). These lower residual displacement experienced by the retrofitted bridge bent would allow the bridge to be in an operational state following a large earthquake. Figure 6.5b shows the residual displacement averaged over 20 ground motions. The similar trend is also seen here, where the ECC jacketed bridge bent suffered lowest average displacement of 61 mm while concrete jacketed bent suffered a maximum average of 113 mm.

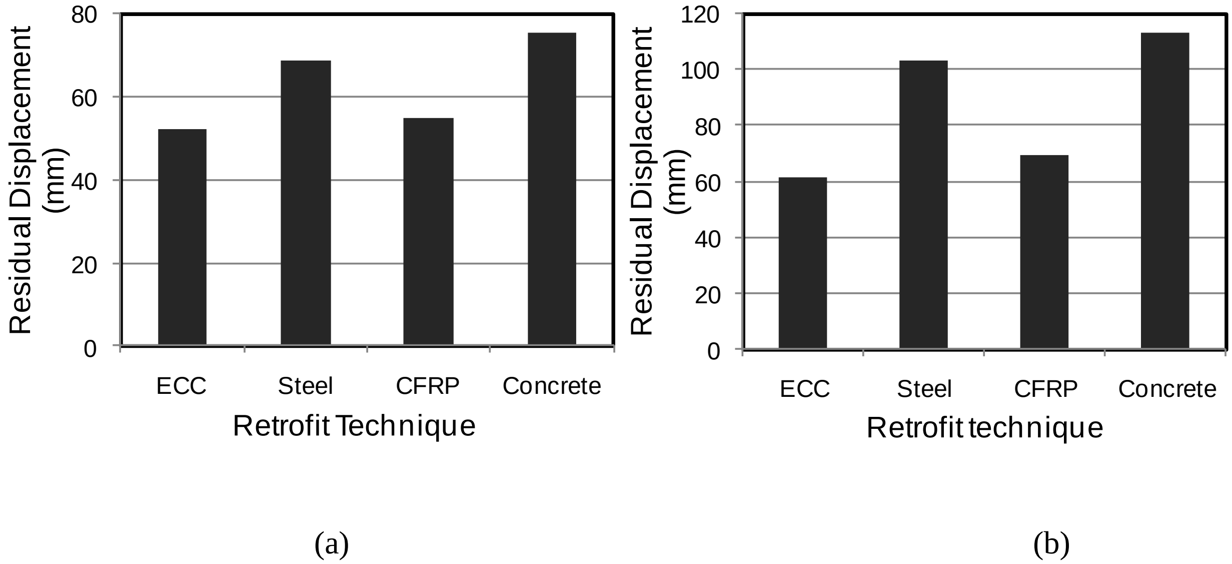


Figure 6.5: Residual displacement comparison of four retrofitting techniques (a) average of ground motion-3 at different intensity level (b) average of 20 ground motion

6.3.2.1.3 Energy Dissipation Capacity

In the case of a reinforced concrete element, energy dissipation capacity during a seismic event proves its structural integrity and shows the interaction between the reinforcement and concrete. The cumulative energy dissipation by various retrofitted bridge bent during seismic excitation was calculated by summing up the dissipated energy in successive load-displacement loops throughout the analysis. Figure 6.6 shows the cumulative energy dissipation capacity of four retrofitted bridge bents under earthquake excitation. Figure 6.6a depicts the average cumulative energy dissipation capacity for ground motion-3 at various intensity levels. From Figure 6.6a it is evident that the steel jacketed bridge bent dissipated larger amount of energy with respect to the other three systems. Concrete jacketed bridge bent dissipated a maximum of 455 kN-m of energy which was 44%, 31% and 15% less than that of steel, ECC and CFRP jacketing, respectively. Figure 6.6b shows the cumulative energy dissipation capacity averaged over 20 ground motions. From Figure 6.6b it is clear that steel jacketed bridge bent dissipated more in all cases as its average energy dissipation capacity is 27%, 13% and 6% higher than that of concrete, CFRP and ECC jacketed bridge bent, respectively.

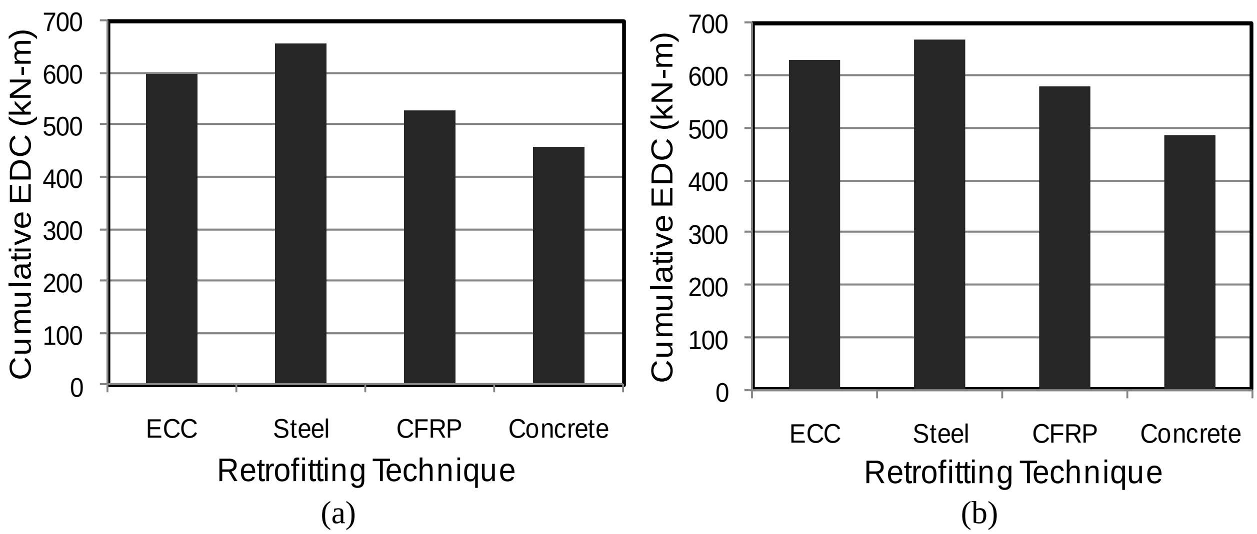


Figure 6.6: Energy dissipation capacity comparison of four retrofitting techniques (a) average of ground motion-3 at different intensity level (b) average of 20 ground motion

6.3.3 Step-3: Weight Generation

After selecting the alternatives and criteria, it is required to determine a set of relative importance (weight) of each criterion. The weights will exaggerate or reduce the evaluations of the alternatives in order to reflect each criterion's importance relative to the others in the choice of the best solution. The Entropy method used herein to compute weights w_i of the criteria C_i (i = 1, 2, 3, 4) is proposed by Hwang and Yoon (1981). This method is particularly suitable for investigating contrasts in discrimination between sets of data. This general concept is widely used in statistical applications exposing unreliability/disorder of a set of data using a discrete probability analysis given a data distribution (Dugdale 1996). Accordingly, it can accommodate many engineering experiments where the input data are obtained within reasonable errors. The use of subjective weighting is avoided here as it requires decision makers experience and judgement.

In this study a weight is determined for each criterion as a measure of its relative importance in a given decision matrix. First the four criteria i.e. base shear capacity demand ratio, ductility capacity, residual displacement and energy dissipation capacity were calculated for the four retrofitted bridge bent to formulate the decision matrix. Only for criteria C_2 (Residual Displacement), the lower the value, the better is the performance of the retrofitting alternative. This is why the inverse of the Residual Displacement is taken in the decision matrix to make its effect positive. Table 6.2 shows the decision matrix for the four alternatives for performance evaluation. In this problem all four criteria are important to choose the best alternative as all four criteria are directly related to the seismic performance of the retrofitted bridge bents.

Table 6.2: Decision matrix

Criteria	C_1	C_2	C_3	C_4
Alternative	Base Shear C/D	Residual Displacement (mm)	Ductility	EDC
ECC Jacketing	2.18	51.96	3.74	608.7
Steel Jacketing	2.02	68.72	3.96	667.5
CFRP Jacketing	2.12	54.69	3.61	577.8
Concrete Jacketing	2.01	75.3	3.46	487.5

Table 6.3: Normalized decision matrix

	Base Shear C/D	Residual Displacement	Ductility	EDC
ECC Jacketing	0.262	0.294	0.253	0.260
Steel Jacketing	0.242	0.223	0.268	0.285
CFRP Jacketing	0.255	0.280	0.244	0.247
Concrete Jacketing	0.241	0.203	0.234	0.208

After the formulation of decision matrix, the normalized decision matrix (Table 6.3) is obtained as follows:

$$P_{ij} = \frac{y_{ij}}{\sum_{i=1}^{n} y_{ij}} \qquad [6.8]$$

The entropy Ej of the set of normalized outcomes of attribute j is given by

$$Ej = -\alpha \sum_{i=1}^{n} p_{ij} \ln p_{ij} \qquad [6.9]$$

Where α is a constant and given as, $\alpha = 1/ln(n)$ [6.10]

E= [E1	E2	E3	E4]
= [0.999	0.992	0.999	0.995]

If no priori weights are given, then the weight of each criterion is calculated as:

$$W_j = \frac{d_j}{\sum_{i=1}^{k} d_j}, \qquad [6.11] \qquad \text{where, } d_j = 1-E_j$$

W= [W1	W2	W3	W4]
= [0.029	0.591	0.062	0.318]

This weight is directly related to the average intrinsic information generated by the given set of feasible alternatives through the j_{th} criterion (attribute) Furthermore; the pie chart in Figure 6.7 represents the shares of importance determined via the entropy method.

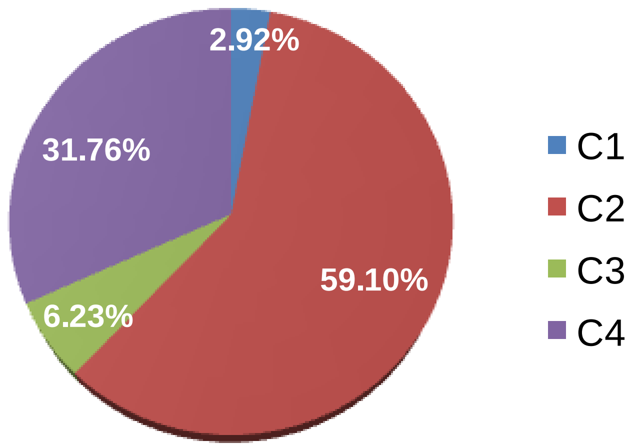


Figure 6.7: Relative importance of criteria

6.3.4 Step-4: Ranking of Retrofitting Alternatives

The ranking of retrofit alternatives were derived using TOPSIS method outlined in Section 2. The ranking of various retrofit alternatives is the last step in deciding the most suitable among available alternatives. The normalized decision matrix and the weighted normalized decision matrix are shown in Tables 6.4 and 6.5, respectively. The two opposite fictitious solutions A^* and A^-, exactly as for the real alternatives A_1, A_2, A_3, and A_4, are defined by four values, representing the performances (weighted and normalized) measured according to the criteria (Table 6.6). The separation of each alternative from *PIS* and *NIS* and the relative closeness C_i^* are reported in Table 6.7. Alternative A_1 (ECC Jacket) resulted as the best one, with a relative closeness, C_1^*, equal to 0.875. Alternative A_1 also results to have the shortest absolute distance from the ideal solution A^* (S_1^* = 0.016) and largest absolute distance from *NIS* A^- (S_1^- = 0.112). So the final ranking becomes A_1, A_3, A_2 and A_4 (Table 6.8). For the investigated case, results (Table 6.8) indicate the ECC jacketing as the final choice. In fact, the low residual displacement and high energy dissipation capacity, which share higher percentage of relative importance of criteria, determined the rank.

Table 6.4: Normalized decision matrix R

	Base Shear C/D	Residual Displacement	Ductility	EDC
ECC Jacketing	0.523	0.582	0.506	0.517
Steel Jacketing	0.485	0.440	0.536	0.567
CFRP Jacketing	0.509	0.553	0.488	0.491
Concrete Jacketing	0.482	0.402	0.468	0.414

Table 6.5: Weighted normalized decision matrix V

	Base Shear C/D	Residual Displacement	Ductility	EDC
ECC Jacketing	0.015	0.344	0.031	0.164
Steel Jacketing	0.014	0.260	0.033	0.180
CFRP Jacketing	0.015	0.327	0.030	0.156
Concrete Jacketing	0.014	0.237	0.029	0.131

Table 6.6: Positive-ideal solution A^* and negative-ideal solution A^-

	C_1	C_2	C_3	C_4
A^*	0.015	0.344	0.033	0.180
A^-	0.014	0.237	0.029	0.131

Table 6.7: Distances S_i^*, S_i^- and relative closeness to the ideal solution C_i^* of each alternative

Alternatives	S^*	S^-	C^*
ECC Jacketing	0.016	0.112	0.875
Steel Jacketing	0.084	0.054	0.390
CFRP Jacketing	0.030	0.093	0.757
Concrete Jacketing	0.117	0.000	0.000

Table 6.8: Rank preference order

Alternative		C_i*	Rank
ECC Jacketing	A_1	0.875	1
Steel Jacketing	A_2	0.390	3
CFRP Jacketing	A_3	0.757	2
Concrete Jacketing	A_4	0.000	4

6.4 SENSITIVITY ANALYSIS

In decision analysis, the final rankings of the competing alternatives are highly dependent on the weights attached to each criterion as the weights attempt to represent the actual significance of the criteria. Small variations in the relative weights can result in a major change in the final ranking. Identification of the most critical criteria can result in a considerable improvement in the decision making process. The common trend is to consider the criteria with the highest weight as the most critical one (Winston, 1991). But this may not always be true and in some instances the criterion with the lowest weight may be the most critical one. Sensitivity analysis is necessary to examine the stability of the ranking under varying criteria weights. In this study the criticality of each criterion is determined by performing a sensitivity analysis on the weights of the criteria (Triantaphyllou, 2002). This method determines the smallest change in the criteria weight that can bring a change in the alternative rankings.

In this study the sensitivity analysis is carried out by examining the degree of sensitivity of each criteria weight determined by entropy method. The smallest change is determined both in absolute and relative terms. Furthermore, it is interesting to investigate whether a change in the current weight value causes any two alternatives to reverse their existing ranking or only the change of the best alternative. Thus, four different sensitivity definitions can be considered.

Herein the Percent-Top (*PT*) and Absolute-Top (*AT*) definition (Triantaphyllou, 2002) is considered, since it is appropriate to survey the best solution changes. The minimum change (according to Triantaphyllou, 2002) for all possible combinations of criteria and pairs of alternatives is shown in Table 6.9. Negative changes in Table 6.9 indicate increasing, while positive changes indicate decreasing. From Table 6.9 it can be easily verified that the *AT* criterion is C_4 (Bold faced). Table 6.10 depicts the percent change in criteria weights obtained by dividing the *AT* value by the weight w_i of criterion C_i. The Percent-Top (*PT*) critical criterion can be found by looking for the smallest relative value in all rows which are related to alternative A_1 (i.e., the best alternative) in Table 6.10. This smallest percentage (i.e., 23.58%) corresponds to criterion C_4 when the pair of alternatives $A_1$and A_2 is considered. In Table 6.9 and 6.10 *NF* stands for non feasible solution (according to Triantaphyllou, 2002). The *PT* value for C_i can also be defined as "*criticality degree*" of the *i*-th criterion. The sensitivity coefficient of the C_i criterion is the reciprocal of its criticality degree. Therefore, the sensitivity coefficients of the four decision criteria are: $sens(C_1)= 0$, $sens(C_2) = 0$, $sens(C_3) = 0.012$, and $sens(C_4) = 0.126$. Therefore, the most sensitive decision criterion is C_4, followed by C_3, C_2, and C_1.

Table 6.9: All possible changes in criteria weights

Pair of alternatives	Criteria			
	C_1	C_2	C_3	C_4
A_1-A_2	NF	NF	-174.408	**-7.48723**
A_1-A_3	NF	NF	NF	NF
A_1-A_4	NF	NF	NF	NF
A_2-A_3	NF	NF	-5.38303	-2.51743
A_2-A_4	NF	NF	NF	NF
A_3-A_4	NF	NF	-16.0268	-10.9599

Table 6.10: All possible percent changes in criteria weights

Pair of alternatives	Criteria			
	C_1	C_2	C_3	C_4
A_1-A_2	NF	NF	-2801.59	**-23.5773**
A_1-A_3	NF	NF	NF	NF
A_1-A_4	NF	NF	NF	NF
A_2-A_3	NF	NF	-86.47	-7.92739
A_2-A_4	NF	NF	NF	NF
A_3-A_4	NF	NF	-257.445	-34.5128

6.5 EFFECT OF NORMALIZATION ON FINAL RANKING

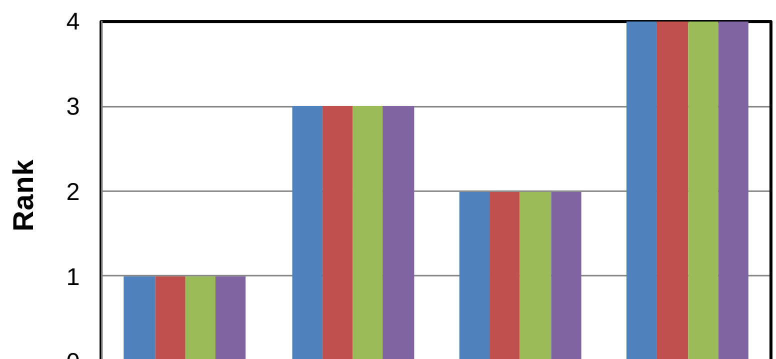
Criteria normalization is an essential step for many compensatory MCDM methods. Normalization refers to the process where multiple sets of data are divided by a common variable in order to negate that variable's effect on the data and allow underlying characteristics of the data sets to be compared (Shelton and Medina, 2010). The purpose of normalization is to obtain comparable scales, which allow inter-and intra-attribute comparisons. If the normalization norm is not applied appropriately, the decision maker may fail to reveal the true decision. Different norms of normalization are available in literature. As the normalization process brings down all the attributes to almost same scale, different normalization approach might have some effect on the final ranking. Many researchers have investigated the impact of the different normalization methods of decision matrix on the decision results (Peldschus 2009; Ginevičius 2008; Zavadskas and Turskis 2008). Investigations by various researchers have proved that the choice of normalization norm may affect the final solution (Zavadskas *et al.*, 2003; Kettani *et al.*, 2004; Saaty, 2006; Opricovic and Tzeng, 2004). Here in this study four different normalization processes have been used in the first step of the TOPSIS method to investigate the effect of normalization. Figure 6.8 shows the different normalization norms adopted in this study. Norm-1 represents vector normalization and the other three norms represent linear normalization. After applying the different normalization process the results are depicted in Figure 6.8. Figure 6.8 also shows the ranking of the alternatives obtained from four different normalization procedures. From Figure 6.8 it is evident that different normalization process yields in the same result. Here in all cases A_1 (ECC Jacket) remains the first choice followed by CFRP, steel and concrete jacket.

Norm-1: $r_{ij} = \frac{x_{ij}}{\sqrt{\sum_{i=1}^{m} x_{ij}^2}}$

Norm-2: $r_{ij} = \frac{x_{ij}}{\sum_{i=1}^{m} x_{ij}}$

Norm-3: $r_{ij} = \frac{x_{ij}}{\max\ x_{ij}}$

Norm-4: $r_{ij} = 1 - \frac{x_{ij}}{\max\ x_{ij}}$



	A1	A2	A3	A4
Norm-1	1	3	2	4
Norm-2	1	3	2	4
Norm-3	1	3	2	4
Norm-4	1	3	2	4

Figure 6.8 : Different normalization norms and their effect in final ranking

6.6 SUMMARY

The MCDM problem of selecting suitable retrofit technique was conducted in this study using TOPSIS method which determined the relative distance from “best” and “worst” solution. The entropy method was adopted herein to determine the relative weights of the conflicting criteria. Both entropy and TOPSIS methods demonstrated a reasonable performance in analysis and in obtaining a solution. A sensitivity analysis was also carried out to determine the most sensitive criteria. The sensitivity analysis revealed the energy dissipation capacity (C_4) as the most sensitive criteria which is true for performance based design as it also represents the ductility of the structure. By introducing different normalizing techniques this study examined the effect of normalization norms on the final decision making. The results demonstrated that the various linear norms adopted in this study do not affect the alternative ranking significantly.

CHAPTER 7 : CONCLUSIONS

7.1 SUMMARY

This thesis proposes a maintenance prioritization methodology for assisting decision makers in resource/fund allocation for bridge rehabilitation. The decision on choosing the most deficient bridge among a number of bridges, which needs immediate repair measures or replacement, has a huge societal and financial impact. A methodology was developed for this purpose considering the existing physical condition of the bridges along with due attention to their economic and social importance. This proposed prioritization technique will certainly help the bridge engineers and decision makers in determining the order of preference for the deserving bridges for repair and rehabilitation. This will help in reducing the number of catastrophic failures, better utilization of public money and high communal living standards through improved safety and security of bridge infrastructures.

This thesis explores the possibility of utilizing different retrofitting techniques for seismic upgrading of a non-seismically designed multi-column bridge bent. This study provides literature review on various retrofitting techniques for bridges; retrofitting techniques available for strengthening of bridge bents and comparative analysis of various techniques have also been presented.

This study demonstrated the seismic performance of a gravity load designed multi column bridge bent retrofitted with different alternatives. A comparative analysis of non linear static pushover and incremental dynamic analysis have been carried out in terms of different performance criteria. This study also evaluates the seismic vulnerability of the retrofitted bridge bents under near fault and far field ground motions. Finally a systematic approach is demonstrated for selection of optimal retrofit technique based on their seismic performances.

7.2 LIMITATIONS OF THIS STUDY

The main limitations of the current study are

- An approximate estimation of the cost of retrofitting.
- Only one type of bridge bent geometry was considered.
- The bridge bents were analyzed considering one set of constitutive models for the materials. Different material model could result in different set of results.
- The near fault and far field ground motions were classified based on their epicentral distance. Other parameters such as *PGA* or *PGV* were not considered to differentiate them.

7.3 CONCLUSIONS

A successful bridge rehabilitation program involves the identification and prioritization of highway bridges in the whole network. Systematic identification of the deteriorated bridges and fund allocation is a crucial problem. The current methods in practice do not permit to follow a systematic approach and establish relation between conflicting criteria for priority ranking of deteriorated bridges. In view of the above facts this study introduces a prioritization method to select the bridges for preventive maintenance and rehabilitation works. In spite of its simplicity, the proposed method offers a systematic assessment of the variables involved and a consistent application of engineering judgment for prioritizing highway bridges. This study considers the influence of social, economic and performance factors on the decision of retrofit prioritization. Successful bridge management and effective fund allocation requires a systematic and robust ranking and prioritization method. The proposed maintenance prioritization method was applied to a specific highway network of British Columbia. This practical application proves that the methodology is capable of handling any number of bridges and applicable for a network of bridges.

The developed research and model constitute a common framework that can be applied to any network of highway bridges in order to prioritize them for retrofitting and maintenance. The proposed *BPI* would be a cost-effective bridge infrastructure rehabilitation decision support tool for engineers and policy makers concerned with bridge management. This study will lead to an efficient solution for bridge infrastructure system and its management.

Current performance based design philosophy requires accurate and simple method for estimating seismic demand on structures considering their full inelastic behaviour. The applicability and accuracy of *SPO* and *IDA* is compared in this study. Moreover this study also compared the performance of various retrofitting techniques for a multi column bridge bent under seismic loading. Based on the results obtained from both static and dynamic analysis the following conclusions are drawn:

- *SPO* heavily underestimates predictions of both crushing displacement and crushing base shear, featuring also excessively low values for other performance criteria. This *SPO* is, therefore, in the opinion of the authors, not adequate for seismic performance assessment of retrofitted bridge bents.

- The results of dynamic analysis show clearly that each earthquake record exhibits its own characteristics. The variation in the results of different ground motions depends on the characteristics of both the retrofitting techniques and the record.
- It is important to analyze bridge structures under high level of shaking where large displacements can occur, and can lead to structural collapse. The *IDA* approach is a systematic method for achieving this end.
- Differences between the crushing displacements estimated by the *SPO* with respect to those by the *IDA* are found to be in the order of 30%, independently of the intensity level of the ground motion.
- The discrepancies between the results obtained from the *SPO* and *IDA* is mainly due to the limitation of *SPO* to predict the higher mode effects in the post elastic region. For all the performance criteria considered in this study, *IDA* predicted a higher capacity range for all retrofitting options in comparison to the results obtained from *SPO*.
- All four retrofit strategies showed good performance under earthquake loading. ECC jacketing retrofitted bridge bent showed higher damage resistance compared to others.
- ECC jacket showed better performance in terms of base shear C/D and residual drift.

From the above discussions it is evident that the most desired retrofitting technique is ECC Jacketing system for multi-column bridge bent because the ECC Jacketing system performed better than any other retrofitting systems considered in this study. The results of the current work seem to indicate that the use of *SPO* analysis might not provide accurate capacity prediction of such retrofitted bridge bents rather a conservative estimate.

This study also developed fragility curves for non-seismically designed bridge bents retrofitted with four different alternatives. The methodology for assessing the fragility of the retrofitted bridge bent includes the use of two dimensional nonlinear analytical models and time history analysis and incorporation of the impact of different retrofit on fragility estimation. Through the process, the impact of retrofit on the probabilistic seismic demand models, vulnerability of the bridge bent is evaluated. The impact of retrofit on *PSDM*s was illustrated to express the shift in ductility demand of the bridge bent resulting from the use of different retrofit measures. The fragility curves for bridge bents are generated using the cloud approach for 20 near-field and 20 far field earthquake ground motion records. The numerical results in general show that bridge bents are more susceptible to the near field seismic ground motions as compared to the far field ground motions. When the bridge bent is subjected to the near-fault ground motion, the ductility demand is very high as expected. The near-fault records produced more vulnerability for the retrofitted bridge bents, which can be attributed to the impulsive effect of near-fault loading. Moreover, the bridge bent retrofitted with concrete jacketing experiences more seismic vulnerability than those retrofitted with the other three jacketing techniques under both near and far field ground motions. In contrary, both the ECC and CFRP jacketed bridge bent possess less vulnerability at all damage state under both near and far field earthquakes. Analyses of the fragility curves reveal that the effectiveness of a retrofit technique in mitigating probable damage is a function of the damage state of interest. The fragility curves as obtained for the bridge bent considered can be used to estimate the potential losses incurred from earthquakes, retrofitting prioritization, post-earthquake rehabilitation decision making and selection of suitable retrofitting techniques.

Selection of an effective retrofit method for seismic upgrading of deficient structures involves a wide variety of criteria and objectives, which demand a formal solution to select the optimum one. An MCDM model can be an effective and powerful tool for engineering decision making problems, in particular when informational (experimental or analytical) data is the only potential measure representing the objectives and constraints numerically. This study aimed at exploring a rational approach to select the optimum retrofit strategy meeting all the performance criteria for retrofitting of a multi-column bridge bent. The benefits of applying MCDM to support decision makers in the selection of effective retrofit strategy based on performance indices for a multi-column bridge bent have been discussed and demonstrated. The proposed methodology is very simple and can be used repeatedly and updated easily with a new set of data for each new project. The results of the investigated case deemed confinement using ECC jacket as the most suitable retrofit option. Moreover, the sensitivity analysis demonstrated a stable result for most critical criteria in both Percent Top and Absolute Top critical criteria. Although the MCDM model presented herein is applied for retrofit selection, it can be applied for any group decision making which is very important for rational decision making procedure.

7.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Further research is necessary to investigate the effectiveness of the proposed prioritization model. This can be done by collecting data of bridges those failed in the past due to their condition deterioration and improper management and applying these data to the proposed model to see whether this model is practical enough to accommodate such bridges. Moreover, a life cycle analysis will definitely make the prioritization procedure a comprehensive one.

Since the present study considers one particular type of bridge bent model without considering uncertainty in geometry and material parameters, a further study using various bridge bent models with different sets of geometry/ material properties should be conducted for better understanding the contributions of other parameters on the seismic fragility of a retrofitted bridge bent. This study examined the fragility response of a retrofitted bridge bent subjected to the horizontal components of the near and far field earthquakes. A further study is required to assess the fragility due to combined horizontal and vertical earthquake motions as this is highly dependent on the phase of the motions.

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