COMPUTATIONAL STRATEGY FOR REHABILITATION OF REINFORCED CONCRETE FRAMED STRUCTURE USING FIBRE REINFORCED POLYMER CONFINEMENT OF COLUMNS

by

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LIST OF ABBREVIATIONS

ADRS	Acceleration Displacement Response Spectrum
ATC	Applied Technology Council
CFFT	Concrete Filled Fiber-reinforced- polymers Tube
CFRP	Carbon Fiber Reinforced Polymer
CSM	Capacity Spectrum Method
DBD	Displacement Based Design
DCB	Deformation –Calculation Based
DDSB	Direct Deformation Specification Based
DL	Damage Limit
DOF	Degree of Freedom
FEM	Finite Element Modelling
FEMA	Federal Emergency Management Agency
FI	Force Index
FRP	Fiber Reinforced Polymer
GFRP	Glass Fiber Reinforced Polymer
HSC	High Strength Concrete
IDSB	Iterative Deformation –Specification Based
JBDPA	Japan Building Disaster Presentation Association
LS	Life Safety
M	Moment
MC	Moment-Curvature
MDF	Multi Degree of Freedom
M-N	Moment- Axial force

MRF	Moment Resisting Frame
NSAP	Nonlinear Static Analysis Procedure
NSP	Nonlinear Static Procedures
PBD	Performance Based Displacement
PI	Performance Index
PMM	Axial force along with moments in two axes of X and Y
RC	Reinforced Concrete
SD	Severe Damage
SDF	Single Degree of Freedom
UBC	Uniform Building Code
VS	Fluid Viscous Dampers
RC	Reinforced Concrete
FRP	Fiber Reinforced Polymer
JBDPA	Japan Building Disaster Presentation Association
ATC	Applied Technology Council
FEMA	Federal Emergency Management Agency
PBD	Performance Based Displacement
FEM	Finite Element Modelling Method
DBD	Displacement Based Design
NSP	Nonlinear Static Procedures
SDF	Single Degree of Freedom
CSM	Capacity Spectrum Method
NSAP	Nonlinear Static Analysis Procedure
DCB	Deformation –Calculation Based

IDSB	Iterative Deformation –Specification Based
DDSB	Direct Deformation Specification Based
GFRP	Glass Fiber Reinforced Polymer
CFFT	Concrete Filled Fiber-reinforced- polymers Tube
HSC	High Strength Concrete
CFRP	Carbon Fiber Reinforced Polymer
MC	Moment-Curvature
MRF	Moment Resisting Frame
ADRS	Acceleration Displacement Response Spectrum
PI	Performance Index
FI	Force Index
M-N	Moment- Axial force
LS	Life Safety
DL	Damage Limit
SD	Severe Damage

LIST OF SYMBOLS

A_c	Cross-sectional area of concrete in column = Ag $(1-\rho_g A_g)$
A_e	Effectively confined area $= A_g - ((h-2r)^2 + (b-2r)^2)/3 - \rho_g A_g$
A_g	Total cross-sectional area
A_s	Area of tensile steel reinforcement
A''_s	Area of compression steel reinforcement
A_{si}	Cross-sectional area of „„ith" layer of longitudinal steel reinforcement
a_p	Spectral acceleration of performance point
a_y	Yield spectral acceleration of bilinear representation of spectral capacity
a	Depth of compression block in RC section
b	Short side dimension of a non-circular cross-section
C_c	Concrete compression force
C_s	Steel compression force
C_n	Sum of columns stiffness in given storey of building
C_A and C_V	Elastic response spectrum coefficients based on seismic sites
d_p	Spectral displacement of performance point
d_y	Yield spectral acceleration of bilinear representation of spectral capacity
D_i	Displacement of top of buildings structure due to lateral push over force
d_{si}	Distance from position of „„ith" layer of longitudinal steel reinforcement to geometric centroid of the cross-section
D	Diameter of circular cross-section or diameter of equivalent circular column for non-circular cross-section column for non-circular cross-sections = $\sqrt{a^2+b^2}$
d	Distance from top of the section to centroid of last tensile steel
e	Eccentricity of axial load
e_{min}	Minimum eccentricity of axial load
E_2	Slope of linear portion of confined stress–strain curve
E_c	Initial modulus of elasticity of concrete
Ef	Tensile modulus of elasticity of FRP
f'_c	Characteristic concrete compressive strength determined from standard cylinder

f'_{cc}	Maximum compressive strength of confined concrete
f^{*}_{fu}	Ultimate tensile strength of FRP
f_{fu}	Design ultimate tensile strength of FRP
f_{l}	Confinement pressure due to FRP jacket
f_{sk}	Stress in „ith" layer of longitudinal steel reinforcement
f_{y}	Yield strength of longitudinal steel reinforcement
f_{yo}	Nominal yield strength for lateral force –deformation
G	Sum of girders stiffness in given storey
G.A	Shear force necessary to cause unit inclination of shear resistance structure
H	Height of column
h	Long side dimension of a non-circular cross-section
M_{max}	Maximum bending moment
Mn	Nominal bending moment capacity
M_{u}	Ultimate moment of RC section
M_{y}	Yield moment of RC section
n	Number of FRP plies composing the jacket
Pa	Constantly applied axial load
Pmax	Maximum applied axial load
Pn	Nominal axial load capacity of a RC column
Q	Lateral shear force due to seismic force
S_{A}	Spectral acceleration of capacity curve
S_{d}	Spectral displacement of capacity curve
SR_{A} and SR_{v}	Seismic response spectrum reduction factors based on damping ratio
r	Corner radius of non-circular cross-sections
t_{f}	FRP nominal ply thickness
T_{S}	Steel tensile force
X	The neutral axis distance from top of the RC section
$^{w_i}/_{g}$	Mass assigned to level i
α	Mean stress factor for confined RC section using FRP
β_{0}	Equivalent viscous damping associated with full hysteresis loop area
β_{Eff}	The effective equivalent viscous damping using for spectral viscous damping reduction factor

β	Linear gradiant coefficient correlated to second part of bilinear shape of spectral capacity
Δ_{demand,c_i}	Required additional displacement of column in order to reach imperative criterions of safe quality
$\Delta_{existent,c_i}$	Existing displacement of column due to lateral push over force
γ_i	The angle in i^{th} step of pushover curve
$\propto_1$	Factor relating the uniform compressive stress of the equivalent rectangular block in the compression zone to the compressive strength $f_c = 0.85$
β_1	Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth
ε_c	Axial compressive strain corresponding to $f_c = 0.002\text{mm/mm}$
ε_{ccu}	Ultimate axial compressive strain of confined concrete
ε_{cu}	Ultimate axial compressive strain of unconfined concrete = 0.003 mm/mm
ε_{fe}	FRP effective strain (strain level reached at failure)
$\varepsilon_{fu\ a}$	Ultimate tensile strain of the FRP
ε_{sy}	Strain corresponding to the yield strength of steel reinforcement
$\varepsilon_{u,frp}$	Rupture tensile strain of the FRP
ε_t	Transition strain in stress–strain curve of FRP-confined concrete. It corresponds to point of change between initial parabola and straight line
	Strength reduction factor
ε_{frp}	FRP Effective strain (strain level reached at failure)
ε_{rup}	Rupture tensile Strain of the FRP
Φ_y	Yield curvature of RC section
Φ_u	Ultimate curvature of RC section
Φ_p	Plastic curvature (${}_{u} - {}_{y}$)
Φ	Angle of inclination based on Stafford theory
Φ_{i1}	Amplitude of mode 1 at level i
ω	Buildings frequency $^{2}/_{T}$
K_ε	Efficiency factor for FRP strain proposed to account for the difference between the actual rupture strain observed in FRP confined specimens, and the FRP material rupture strain determined from tensile coupon testing
ρ_g	Ratio of the area of longitudinal steel reinforcement to the cross-sectional area of a compression member

K_a	Efficiency factor for FRP reinforcement in the determination of f'_{cu} (based on the geometry of the cross-section)
K_b	Efficiency factor for FRP reinforcement in the determination of ε_{cu} (based on the geometry of the cross-section)
$_f$	Additional FRP strength reduction factor
$_y$	Yield curvature of RC section
$_u$	Ultimate curvature of RC section
ϕ_{Target}	Target curvature of collapsed column to reach the desired displacement
Z	Seismic zone factor mentioned from Table 16-I of the UBC
L	Column length
I	Importance factor from Table 16-K of UBC code
P_0	Constantly applied axial load
R_w	Response modification factor (R) from Table 16-N or 16-P of the UBC
W	Total weight of the structure in seismic form.
V	The lateral force due to given earthquake, the base shear of structure
C	Earthquake force coefficient multiplying to building weight (W)
$A \times B$	Design spectral acceleration, is known as the ratio of gravitational acceleration corresponding to fundamental period of structure T and also soil type
R	Response modification factor, for specific structural system from UBC and 2800 buildings code
R_μ	Ductility base reduction factor
Δ_{max}	Maximum displacement against the lateral force
Δ_y	Yield displacement
T_g	Predominant period of ground motion
T	Natural period of structure
$\varphi_{ed}, \varphi_{ev}$	Amplification factor

STRATEGI PENGIRAAN UNTUK PEMULIHAN STRUKTUR KERANGKA KONKRIT BERTETULANG MENGGUNAKAN TIANG TERKURUNG POLIMER BERTETULANG GENTIAN

Abstrak

Pemulihan seismik untuk struktur konkrit bertetulang (RC) menggunakan Polimer Bertetulang Gentian (FRP) untuk tiang terkurung telah digunakan dengan meluas. Walau bagaimanapun, tindak balas global struktur yang dipulihkan dengan menggunakan Rekabentuk Berpandukan Pesongan tidak dikaji sepenuhnya. Tambahan pula, pengurangan dalam penggunaan bahan dan masa analisis perlu ditimbangkan. Oleh itu, kajian ini dijalankan untuk menyiasat tindak balas keseluruhan struktur RC yang dipasang tiang terkurung FRP apabila analisis tidak lelulus digunakan untuk menilai struktur. Keputusan analisis untuk tiang terkurung FRP disahkan terlebih dahulu dengan menggunakan keputusan lapan belas eksperimen. Ini diikuti oleh kajian tindak balas global struktur yang terdiri daripada tiga ke tujuh tingkat. Tindak balas global struktur kerangka yang dipulihkan dengan tiang terkurung FRP didapati sama tanpa perubahan kekukuhan di sampul pasca-alah dalam lengkung “pushover” konvensional dan ADRS. Tindak balas kerangka konkrit bertetulang yang dipulihkan disahkan terlebih dahulu dengan penyelesaian analitik. Satu prosedur yang sistematik kemudiannya dicadangkan untuk pemulihan seismic struktur berkerangka RC menggunakan tiang terkurung FRP berdasarkan kaedah sistem penilaian lelurus dan tidak lelurus. Kaedah tidak lelurus menunjukkan penggunaan bahan yang kurang daripada kaedah lelurus. Akan tetapi, masa pengiraan bertambah dengan ketara. Keputusan kajian ini menunjukkan prosedur sistematik pemulihan untuk kerangka konkrit bertetulang dengan FRP yang dicadangkan mampu mencapai objektif pencapaian dengan penggunaan bahan dan masa analisis yang kurang.

COMPUTATIONAL STRATEGY FOR REHABILITATION OF REINFORCED CONCRETE FRAMED STRUCTURE USING FIBRE REINFORCED POLYMER CONFINEMENT OF COLUMNS

ABSTRACT

Fibre Reinforced Polymer (FRP) has been widely used for retrofitting of reinforced concrete (RC) columns. However, the global response of retrofitted RC framed structure based on Displacement Based Design (DBD) method is not clearly known. In addition, reducing consumption of material and analysis time should be taken into account. Thus, this study is aimed to determine the global response of retrofitted structure using FRP-confined columns when nonlinear analysis is used to assess the structure. The analysis results for FRP-confined RC columns were first verified using eighteen experimental results found in literature. The analysis was then proceeded to the whole RC frame consisting of three to seven storey levels. The global response of retrofitted RC frame using FRP-confined columns has been found to be identical in post-yield envelope at pushover curve in conventional and ADRS formats without changing in initial stiffness. Response of retrofitted RC frame was firstly verified using analytical solution. A systematic procedure was then developed for seismic rehabilitation of RC frame based on both linear and nonlinear methods of assessment. Four case studies were used to demonstrate the applicability of proposed systematic procedures for linear and nonlinear methods of assessment. It was found that retrofitting using nonlinear method give less material consumption as compared to linear method. However, the computational time increases substantially. Results of the current study have demonstrated that the proposed systematic rehabilitation procedure for RC framed structure using FRP is capable to reach its performance objective using less material as well as computational time.

A CHAPTER 1

INTRODUCTION

1.1 Background

Recent studies have shown that reinforced concrete (RC) buildings without adequate reinforcement are inherently brittle during earthquakes (Pampanin et al., 2002; Hart, 2005; Galasso et al., 2010; Pampanin et al., 2011). The inadequate resistance of building against earthquake and gravity loads may result in failure of RC columns and consequently the collapse of buildings and loss of human life. RC failure may occur because of shortage of stiffness or displacement capacity of RC columns against combination of earthquake and gravity loads. Figure 1.1 shows the inadequacy of transverse steel reinforcement at beam column joints subjected to simultaneous axial and lateral force resulting in loss of capacity of the RC column.

Figure 1.1: Failure of RC columns due to earthquake (Canfield and Sabrina, 2010)

Low displacement capacity of the RC column at the local aspect results in the formation of undesirable soft-stories and greatly affects the overall structural performance. To avoid such tragedy, structures must be assessed for their ability to withstand against seismic loads. Structural assessment is divided into two subdivision: force control and displacement control. Conventionally, seismic assessment and rehabilitation of existing structures have been carried out by using the force-based approach. In this method, a combination of gravity and lateral force due to earthquake is applied to the structures and consequent forces in members are estimated. In the last decade displacement-based methods have emerged as a favoured alternative because displacement-based parameters can quantify structural and non-structural damage better than force-based parameters (Priestley, 1997; Sullivan et al., 2004; Oğuz, 2005; Faella et al., 2008). Although the stress ratio of structural members is defined as a criterion in force control codes, displacement is the main criterion used in displacement-based design (DBD) codes for assessing building structures.

The current procedure of assessment based on displacement based-design (DBD) codes such as ATC-40 is shown in Figure 1.2. As shown in Figure 1.2, the capacity curve can provide insight to engineers to make them aware about the condition of the structure in terms of its elastic and inelastic parts by comparing the parts with the demand of the earthquake curve.

Capacity of structure is plotted with earthquake demand to determine visually the structure ability in elastic and inelastic segment. Structure capacity is defined as the displacement of structure, corresponding to lateral force plotted in the pushover curve. Seismic demand is the maximum identified damage caused by seismic force.

Thus, performance thus is defined based on the displacement of a structure against lateral force due to earthquake. The overall goal of performance-based design (PBD) methods is to develop structures that can satisfy the desired performance objectives and consequently achieve the desired life-safe quality.

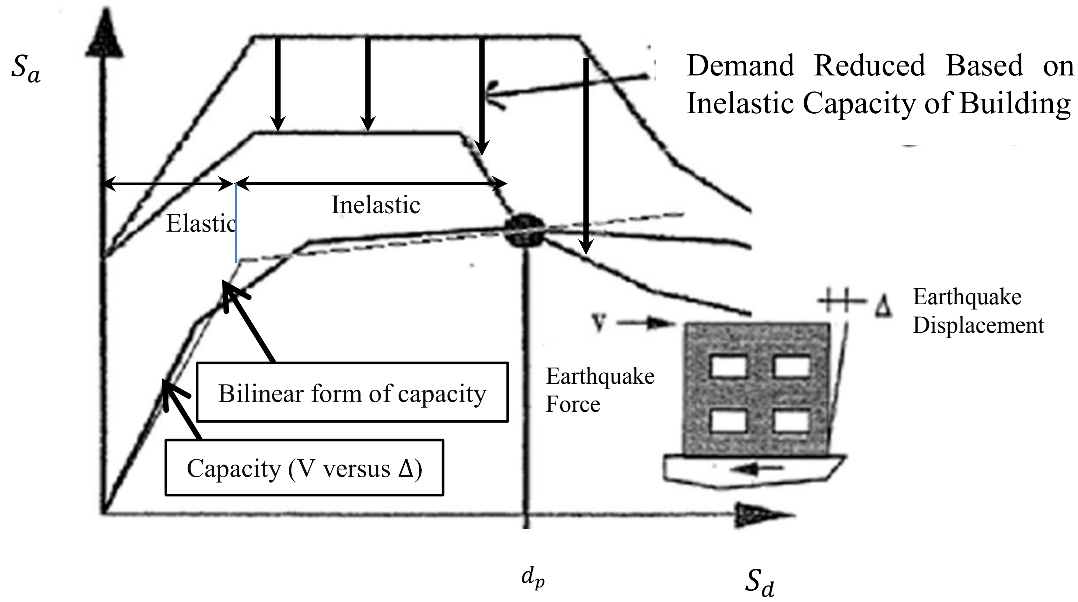


Figure 1.2: Assessment of structures (ATC-40)

Engineers and designers have to develop their skills to evaluate the performance of buildings by using recent methods of assessment, such as PBD. The Performance Based Design (PBD) method has become the most common method for evaluating RC frames using nonlinear analysis on the basis of displacement-based design (DBD) (ATC40, 1996; Sullivan et al., 2003). The rationale behind displacement-based methods is that; contrary to force-based methods, DBD does not consider earthquake force as a set of lateral force in the seismic design. Displacement of structure against lateral force is used as criterion for evaluating and assessing safe quality of structures (Priestley, 1997).

In PBD methods, the acceptable performance is calculated by measuring the level of structural and non-structural damage levels. Damage is expressed in terms of inelastic deformation limit of RC frame that is called post-yield behaviour of the RC frame in pushover analysis. The analysis procedure concerning post-yield behaviour of structures is more complex compared with that of traditionally used in the force-based design process for evaluating performance of structure against earthquake (FEMA440, 2004; Oğuz, 2005).

Currently, seismic assessment based on performance based design (PBD) consists of collecting the as-built information and analysing the structure to obtain the current situation of structure. The DBD method is known as the most recent tool for assessing structural performance against earthquake (Sullivan et al., 2003; Sullivan et al., 2004; Kim and Choi, 2006). The performance of structures according to ATC-40 (1996) for structural level is defined in six groups, namely, immediate occupancy, damage control, life safety, limited safety, structural stability and the not considered that as shown in Figure 1.3. They are defined based on displacement of structure.

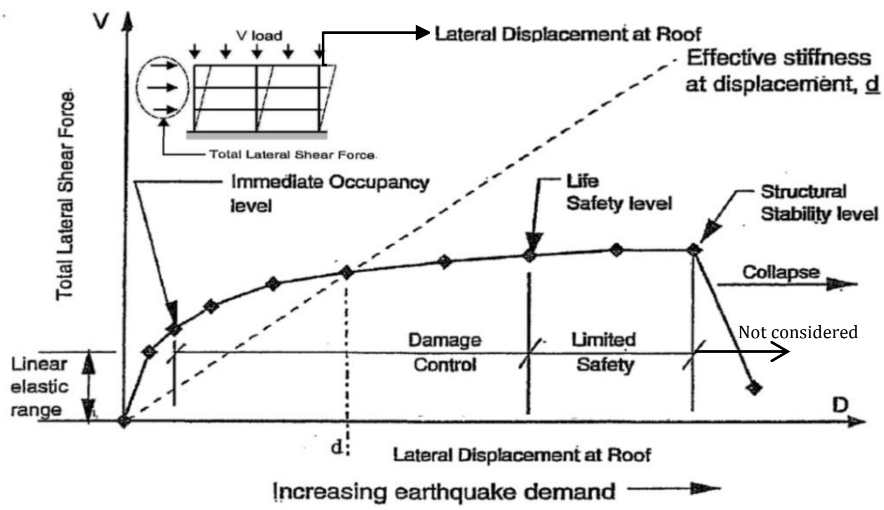


Figure 1.3: Defining performance level based on displacement capacity (ATC-40)

To assess the resistance of a structure against earthquake, many performance objectives are introduced to analytically predict the performance of structure against the combination of earthquake and gravity loads. For example, stress ratio of columns is the performance objective in code such as uniform building code (UBC) which is used to assesses structures on the basis of linear analysis methods. The column stress ratio (CSR) is defined as the ratio between stress on the top of the column caused by axial and moment forces to the average applied embankment stress at the level of the top of the column. The axial and moment are caused by the combination of gravity and earthquake loads. It is the factor that specifies the desired seismic performance of the RC frame. Performance objective determines the ideal seismic performance of RC frame. Seismic performance is described by designating the maximum permitted damage status (performance level) for a particular seismic hazard (earthquake ground motion).

On the other hand, displacement capacity of a structure up to having performance point is one of the performance objectives according to ATC-40 that follows nonlinear method of analysis. The procedure of defining performance point is shown in Figure 1.4 where the performance point is known as intersection of earthquake demand to capacity in spectral form.

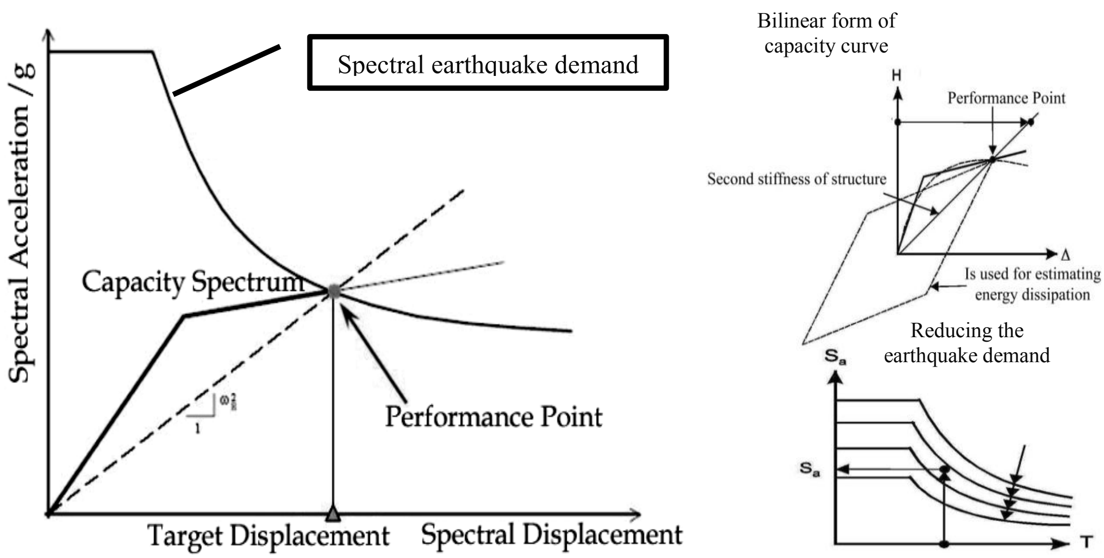


Figure 1.4: Performacnce point determination (ATC-40, 1996)

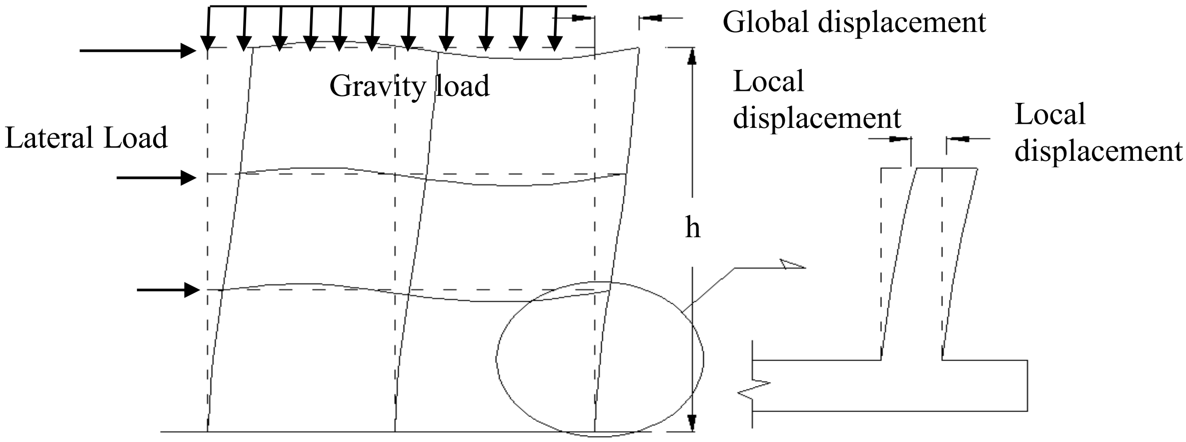
Demand is produced based on characteristics of earthquake in spectral form and is reduced corresponding to ductility of structure. For reducing the demand earthquake, capacity curve as shown in Figure 1.4 is bilinearized and damping ratio for reducing the demand based on characteristics of bilinear form of capacity is calculated.

Structures that do not comply with the design codes criterions (i.e., having performance objectives) should be retrofitted so that the deficiency of weak structural members with insufficient force or displacement capacity can be rectified. The performance objective has to be determined according to the building owner"s decisions for safe quality classifications and performance level. After the owner selects the performance level, the engineer will develop a sufficient plan to reach the desired performance objective.

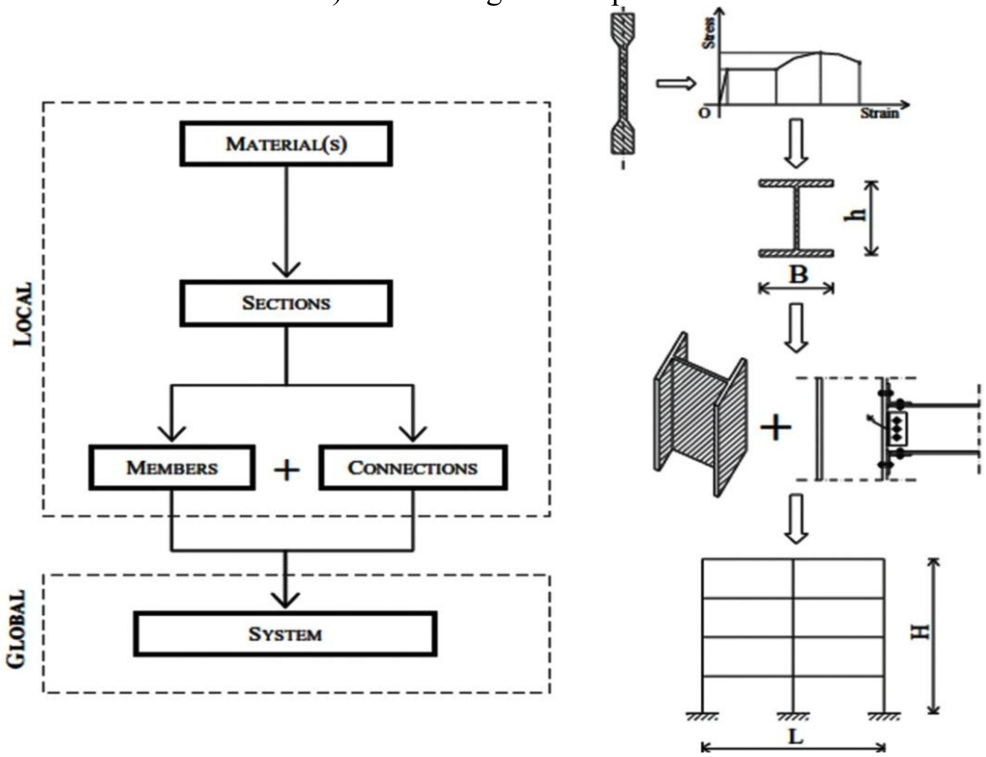
Building deficiencies may be caused by changes in the occupancy of the building, occurrence of earthquakes, errors of contractors and designers, and alterations in geotechnical resistance. These deficiencies lower the strength and ductility of the structure, thus placing the life of building occupants at risk. To avoid tragedies, various types of rehabilitation methods have been proposed during the last two decades (Thermou and Elnashai, 2006). Engineers and researchers have to determine the optimum solution in terms of time, cost, and structural integrity in selecting appropriate rehabilitation techniques. The decision making process for RC frame retrofitting depends on the experience of engineers and designers. Not only the local behaviour of retrofitted members should be known, the impact of response of local retrofitted member on global response of RC frame also has to be recognized before making decisions. This is because both local and global responses such as displacement or internal forces are used to assess the structures. When evaluating structures, engineers and researchers should not only consider the local response of retrofitted structure but also the global response of structures. For example, brittle mechanism of column can be developed from local (captive or short columns) to global level (soft-storey) (Kam et al., 2011). As can be seen in Figure 1.5, local deficiency of RC column may result in global collapse of RC frame.

Figure 1.5: Effect of local behaviour of structural member on global response of RC frame (Kam et al., 2011)

In Figure 1.6 (a), displacement of a column is local displacement and the displacement of the entire structure is known as global displacement. Local response of frame as shown in Figure 1.6 (b) refers to response of material, section, type of structural member and connection between members. Global response of structure is the response of the RC frame such as displacement or drift of frame against applied force due to lateral and gravity loads.



a) Local and global displacement



b) Hierarchical relationship between local and global structural response (Elnashai and Di Sarno, 2008)

Figure 1.6: Local and global response of structure

Local response of members in structure affects global response of structure. For example by increasing displacement capacity through changing material or section of structural member, displacement capacity of frame is consequently increased.

In the current PBD evaluation processes, the analytical models of structure through a full range of global and local displacements must represent complete characteristics of RC frame behaviour, including mass distribution, strength, stiffness and deformability. The consequences of elastic and inelastic response should be considered at both local and global levels to ensure the safety of structures (ATC40, 1996; FEMA356, 2000).

Enhancing the global capacity of the retrofitted structure up to the expected levels of stiffness and ductility can be conducted through the increment of local capacity of structural members. The stiffness and ductility of the structure are the functions of the structural member stiffness and ductility and any changes in stiffness of structural member such as size increment, will change the stiffness of entire structure. The stiffness of the whole structure is obtained by dividing lateral force to its corresponding displacement. The calculation process of structure stiffness involves complex computation. The ductility increment of the local RC member during retrofitting will change the entire ductile behaviour of the structure. Thus, the retrofitting approach for seismic enhancement should focus on the required capacity of structure against the seismic-induced displacement demand. The increment of ductility of the structural member should consider the global flexibility of the structure. In addition, the lateral force resisting system of the existing structure should be taken into consideration. As a result, the displacement of the retrofitted structure ranges within the limits of the displacement capacity of the existing structure.

Many types of rehabilitation, such as using shear walls, steel braces, steel jacketing and concrete jacketing are available for increasing stiffness of structure. Other types of retrofitting, such as using FRP jacketing of columns, viscous dampers, and isolators, are adopted to increase the displacement capacity of members and structure. Generally, after selecting the type of retrofit for overcoming deficiency of structural members and consequently the entire structure, the amount of material and size of additional retrofit elements must satisfy performance objectives in codes. The conventional method of rehabilitation does not take into account of the optimum amount of material needed by the structural member in RC frames. The method relies on the experience, creativity, capability, and intuition of decision makers and designers. In the absence of any systematic method, the conventional method adopts trial-and-error approach for rehabilitating elements of frames to select the size and location of retrofit element. This approach may cause ineffectiveness in terms of material usage for rehabilitation. For example, using FRP for columns in order to overcome the problem of lack of ductility of structures and reach desired performance objective. In this case, to achieve desired performance objective that contained specified ductility of structure based on codes, emplacement of FRP-confined columns and amount of material in the absence of any systematic method must be selected by using trial-and-error approach. To solve this problem, various studies have proposed the use of the systematic methods for eliminating problems arising from conventional approach of rehabilitation for selecting the location and amount of material of retrofit elements. Previous studies have systemically selected the stiffness characteristics of nonlinear fluid viscous damper (VS) (Martinez-

Rodrigo and Romero, 2003), amount of FRP and emplacement at weak joints at RC beam and columns joint (Pampanin et al., 2007) and geometric characteristics and stiffness of haunches at RC beam and columns joints (Pampanin and Christopoulos, 2003). These systematic methods aimed to select amount, location and characteristics of retrofit elements to maximize the performance of structural components while minimize the consumption of computational time and materials.

FRP is one of the popular materials for structural rehabilitation. The advantages of FRP are the light weight, ease of application on RC columns, and noncorrosiveness. However this material is costly. Although the advantage of this material for enhancing shear and deformation of RC columns is proven (El-Sokkary and Galal, 2009), nevertheless global response of structure retrofitted by FRP-confined columns is not known in DBD methods of RC frame assessment. An ideal retrofit strategy for RC structures not only protect the column, by identifying damaged columns and applying FRP on weak columns, but also further upgrade the RC frame to exhibit the desired behaviour according to performance objectives specified in codes.

1.2 Problem Statement

The performance of structure is affected because of changes in the occupancy of the building, occurrence of earthquakes, errors of contractors and designers, and alterations in geotechnical resistance. For assessing performance of RC frames, every code has introduced specified range of performance objectives that must be satisfied to ensure safe quality. The values for performance objectives are obtained by analysing RC frame. Performance objectives include the specified range of both local and global response of RC frame against seismic and gravity forces. Buildings with shortage of specified values of performance objective must be retrofitted.

For retrofitting RC frame, the first stage is the selection of the appropriate retrofit strategy. Thus the best decision must be made concerning cost and weakness of structure. For selecting the most rational retrofit strategy, decision makers and designer teams must be clearly aware about impact of retrofit strategy on given performance objectives. Otherwise, retrofit plan may lead to inappropriate response of structure and undesired performance objectives.

Although FRP studies have reported elimination of shear failure of retrofitted RC column (Thermou and Elnashai, 2006), the global response of RC retrofitted frame has not been studied based on PBD methods of assessment. Although, confining RC columns by FRP has impact on local response of structural member as increment of ductility of RC member without considerable effect on stiffness (Balsamo et al., 2005; Zou et al., 2007; El-Sokkary and Galal, 2009), global response of RC frame is still not known in PBD method of RC frame assessment. Therefore impact of using FRP-confined columns on global response of RC frame when assessed based on PBD method of assessment is not known. Thus, for researcher and practical engineer, effect of FRP on specified global performance objectives specified in ATC-40 is unknown. ATC-40 is the code using displacement for assessing performance of RC frame (PBD) and displacement is obtained using nonlinear method of analysis that is inherently complex and time-consuming (ATC-40, 1996; FEMA440, 2004; Oğuz, 2005).

Although having adequate information about the effect of FRP-confining columns on performance objectives aids designers to appropriately make decision but, emplacement and amount of material to achieve ideal performance objectives according to ATC-40 and UBC-97, is conventionally determined following conventional trial-and-error approach since there are no systematic methods. Apart from structural response of retrofitted structure using FRP-confined columns, systematic method to rehabilitate structures using the optimum material and analysis time to achieve the required performance objectives according to ATC-40 and UBC-97 is still not available. Although Zou et al. (2007) has optimized FRP consumption for RC frame using complex mathematical procedure, the method is based on performance objectives specified in Chinese code. Emplacement of FRP-confined columns has not been considered and FRP must be applied to all columns of RC frame which is costing. Owner of buildings will not accept demolishing all non-structural elements of building such as infill walls and HVAC that will cost more than reconstruction of the building. Meanwhile moment-curvature (MC) diagram of FRP-confined columns that determines displacement capacity of column in study by Zou et al. (2007) had not been verified. Global response of RC frame after retrofitting is not reliable, without verification of local response of RC frame.

The analysis process according to ATC-40 and UBC-97 codes usually needs a linear and nonlinear static analysis of structure that are considerably time-consuming. Therefore, developing a systematic procedure for selecting the amount of material and location of FRP-confined columns is needed to reduce computational efforts and to make seismic assessment more reliable.

1.3 Objectives

The objectives of this study are as follows:

i) To study the local and global structural response of RC frames after retrofitting with FRP-confined columns

ii) To propose a systematic procedure for selecting the proper location of FRP-confined columns and number of FRP layers for RC column on the basis of linear and nonlinear methods of structural assessment

iii) To assess the performance of the proposed systematic methods in terms of time and material consumption.

1.4 Scope of Study

This study focuses on the seismic rehabilitation of rigid RC-framed with FRP-confined columns. Pushover nonlinear static analysis is adopted to evaluate the safety level of RC frames. Structural assessment is carried out based on nonlinear and linear methods (ATC-40 and UBC97). To verify the analysis results for FRP-confined columns, 18 experimental results are compared with the analysis results from SAP2000 modeling.

The case studies consisted of 3D frames for investigating the global response of retrofitted structures and 2D frame for evaluating the proposed systematic methods. 3, 4, 5, 6 and 7 stories RC frames were analysed in pushover analysis. The cross sections of structural members in the case studies were limited to rectangular.

SAP2000 was used to analyse the behaviour of RC frames. In this study, the confined concrete material properties are defined according to Lam and Teng (2003a and 2003b). ACI440 is used to define the behaviour of confined concrete to limit the rupture strain of FRP. Thus, columns of RC frames were modeled with and without consideration to ACI440 limitation for rupture strain. RC frames were simulated with assumption that hinges are formed at both ends of columns and beams.

The variables used in this study to reach performance objectives were the number of FRP layers and emplacement of FRP-confined columns. Performance point is denoted as the performance objective according to ATC-40, whereas CSR is denoted as the performance objective based on UBC97.

1.5 Structure of Thesis

This thesis is divided into six chapters.

Chapter 1 introduces the research background, problem statement, objectives of research and scope of work. Chapter 2 reviews the background of seismic rehabilitation. Experimental and numerical researches on FRP-confined columns were then reviewed. Rehabilitation methods and consequent local and global response of structure using various retrofit elements is then described. It is followed by review on systematic methods of rehabilitation using FRP.

Chapter 3 presents the theoretical background and basic formulations for behaviour of FRP-confined columns. The processes of assessment of RC frames based on linear and nonlinear methods are described and all formulations that are used in this study are presented. The formulations used for determining displacement of RC rigid frame against lateral force are also mentioned.

Chapter 4 describes the research methodology used in this study. The procedure of modelling FRP-confined columns to compare to experiments from previous studies is explained. Then systematic methods based on linear and nonlinear methods of assessment of RC frames are described and illustrated in terms of flow chart and scheme of algorithm.

Chapter 5 presents the result of analysis and discussion. The analysis results for FRP-confined columns are verified using 18 experiment columns that had been studied before. The results of comparison of analysis and experimental results are presented. Whole retrofitted structures are modelled and global response of retrofitted RC frame is shown and discussed. The global response of retrofitted RC frame is verified using theoretical solution. The applicability of proposed systematic method based on nonlinear and linear methods of assessment are evaluated using four case studies. Advantages of proposed systematic method are then highlighted by comparing to other similar study on optimization of FRP consumption for retrofitting RC frames.

Finally the conclusion and recommendation for future research work are presented in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter is divided into four sections. The general procedure of seismic rehabilitation and analysis methods is first presented. The effect of retrofit on the local and global response of RC frame by using FRP is described. Subsequently, reviews on related past studies on FRP-confined columns are presented, and the systematic method to achieve cost-effective solution in seismic rehabilitation is depicted.

2.2 Seismic Rehabilitation Procedure

Many recent codes, such as SEAOC (1999), ATC-40 (1996), FEMA 356 (2000), FEMA 440 (2002), and FEMA P695 (2009), that define the performance of structures on the basis of displacement are used to evaluate structures. Some of aforementioned code such as ATC-40 (1996), FEMA 440 (2002) employ the capacity spectrum method (CSM) in first and second stiffness approaches to analyse and assess the performance of RC frames. In the CSM proposed by Freeman (1998), the capacity in spectral form has to be linearized to obtain the required parameters, such as performance point and reduction factor.

The role of analysis methods in obtaining accurate displacement of the RC frame against force has attracted much attention. Table 2.1 lists the research studies on the calculation of the displacement of structures against forces. In Table 2.1, the horizontal row shows the displacement method used in the design process, whereas the vertical column shows the analysis method. ATC-40 is generally in the category of the methods of Freeman (Freeman, 1998), Paret et al. (1996), and Chopra and Goel (2001) that are based on CSM. However, other methods have also adopted the secant stiffness for structure assessment and analysis (Gulkan and Sozen, 1974; Priestley, 1993). The base method of guidelines, such as ATC-40, involves the calculation of maximum displacement expected from a given structure. DBD methods which are based on initial stiffness determine the displacement of structures by using elastic stiffness of structure and approximate the relation between elastic and inelastic responses.

Table 2.1: Design procedures for structure assessment through Displacement Based Design Methods (Sullivan et al., 2003)

	Deformation – Calculation Based (DCB)	Iterative Deformation – Specification Based (IDSB)	Direct Deformation Specification Based (DDSB)
Response Spectra: Initial Stiffness Based	Moehele (1992) FEMA (2000) UBC (1997) Panagiotakos and Fardis (1999) Albanesi et al. (2000) Fajfar (2000)	Browning (Browning, 2001)	SEAOC (1999) Aschheim and Black (2000) Chopra and Goel (2001)
Response Spectra: Secant Stiffness Based	Freeman (1978) Paret et al. (1996) Chopra and Goel (1999) ATC-40 (1996)	Gulkan and Sozen (1974)	Kowalsky et al.(1995) SEAOC (1999) Priestly and Kowalsky (1998)
Direct Integration: Time Historey Analysis Based	Koppas and Manafpour (2001)	N/A	N/A

Figure 2.1 illustrates the meaning of initial and secant stiffness. The dynamic information of structures can be obtained from the elastic behaviour of structure in the initial stiffness method by approximating the elastic and inelastic responses of the structure.

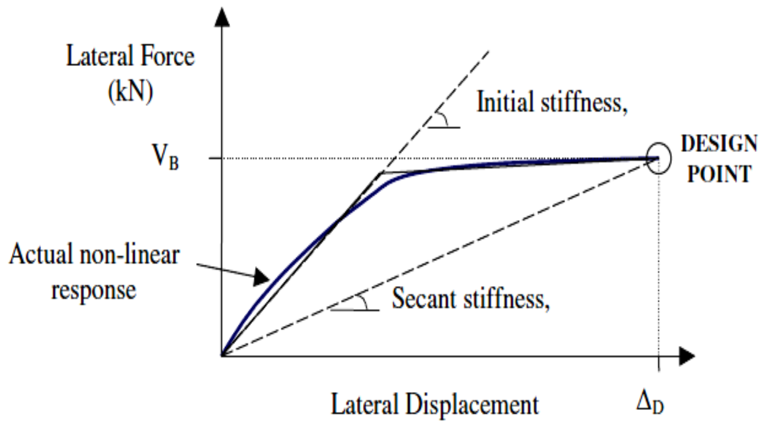


Figure 2.1: Initial and secant stiffness (Sullivan et al., 2003)

The flowcharts of retrofitting process based on ATC-40 and FEMA 356 are shown in Figure 2.2 and Figure 2.3, respectively. A retrofit plan is first assumed in this process. The resulting capacity and demand spectrum is developed, and the performance point is determined. If the results are in the acceptable performance objective range, then the design is adequate. If not, the design must be revised by changing the placement and sizes of retrofit elements. The process is subsequently repeated, or an alternative strategy is employed (ATC40, 1996). Thus, to achieve the desired performance level and performance objectives of a structure, a formulation of rational retrofit plan must be considered.

To reach this aim, the design team must be aware about the effect of any possible retrofit strategy on performance objectives. Otherwise, the highlighted stages in Figure 2.2 and Figure 2.3 must be performed by a conventional trial-and-error approach. The formulation of retrofit strategies requires experienced decision makers. Without any clear information on the effect of possible retrofit strategies on performance objectives, developing any plan for retrofitting may lead to undesired values. If the designer does not know the exact local and global response of the structure after rehabilitation by using possible retrofit strategies, the developed plan may not be able to overcome the structure deficiency.

To achieve the desired performance objectives, the placement and determination of sizes of retrofit element and material consumption must be initially decided after selecting the retrofit element. The selection of placement and sizes of retrofit element must be revised or altered if the desired performance objective is not achieved (ATC-40).

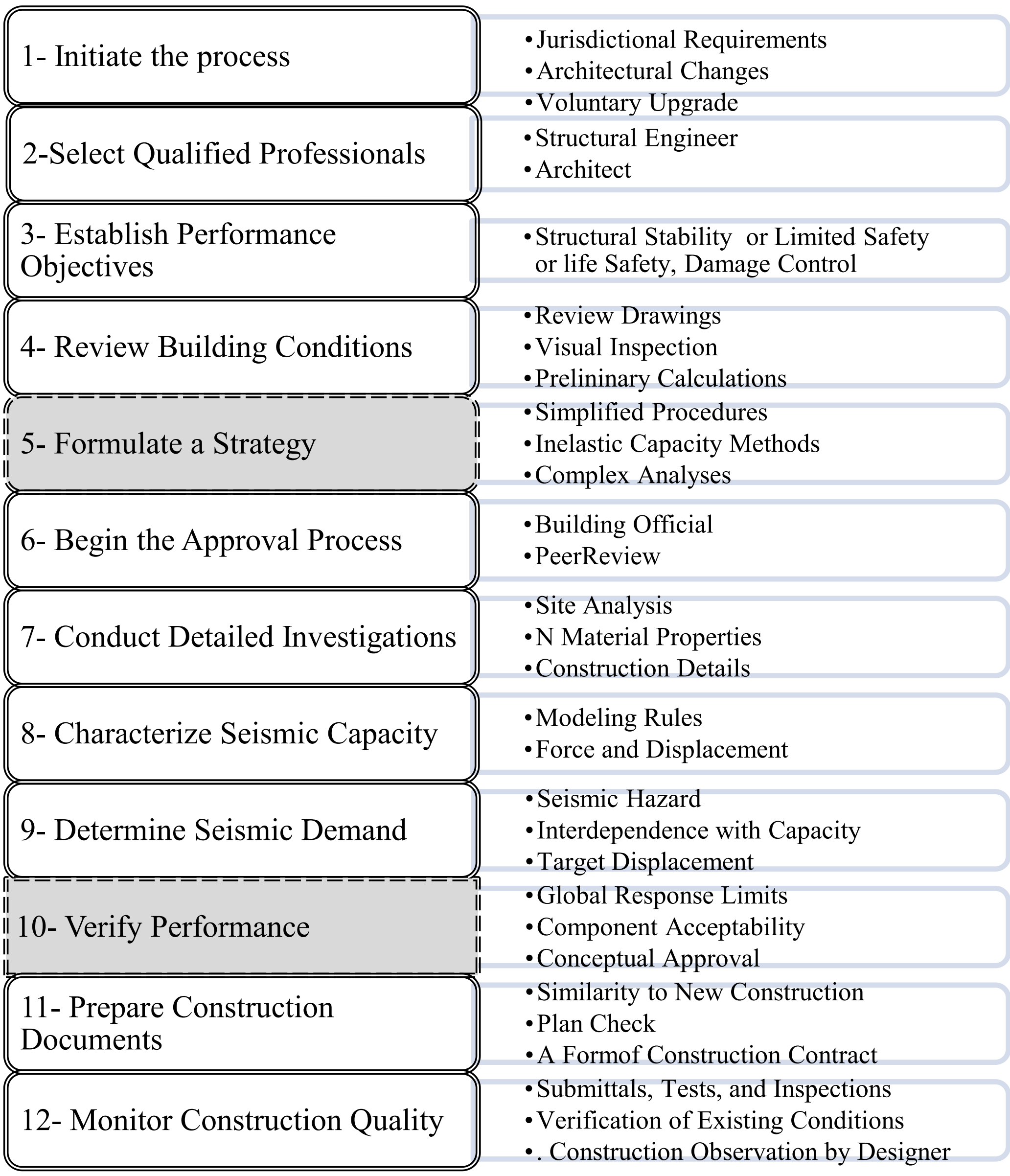


Figure 2.2: The process of rehabilitation according to ATC-40 (1996)

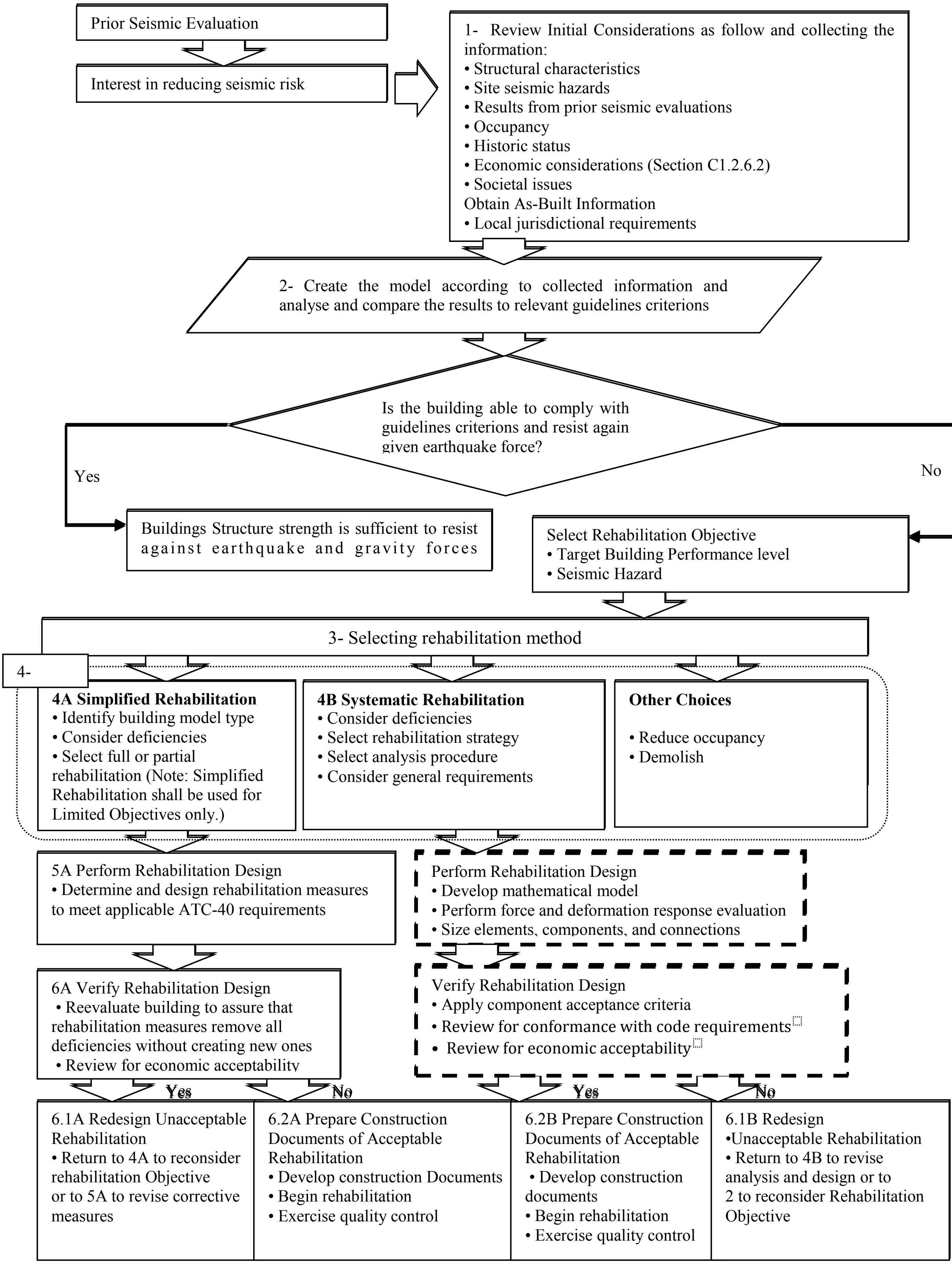


Figure 2.3: Rehabilitation process flowchart (FEMA356, 2000)

In order to achieve desired performance objectives, placement and determination of sizes of retrofit element and material consumption must be initially decided in first step after selecting retrofit element. The selection of location and sizes of retrofit element must be revised or altered if desired performance objective is not achieved.

2.3 Numerical Modelling of FRP-Confined RC Column

Predicting the behaviour of FRP-confined columns against the lateral and gravity loads has attracted the attention of researchers. Researchers not only attempted to predict the behaviour of FRP-confined columns by using various experiments but also modeled FRP-confined columns numerically to analytically estimate column responses against forces. Koksal et al. (2009) studied the axial force modeling of the FRP-confined column by using the cohesion parameter of Drucker–Prager criterion in terms of cylindrical compressive strength to predict the compressive performance of concrete under high confinement pressures. Koksal et al. (2009) also modeled 42 small- and large-scale column specimens tested by eight different researchers. Analysis results have achieved adequate accuracy for axial stress-strain relation of FRP-confined RC column. Figure 2.4 shows the finite element model of the column.

Figure 2.4 shows that eight- and four-point elements were used for solid elements of concrete and FRP. The top surface of the element is modeled in a rigid form. An eight-node concrete in 3D form and isoperimetric (isotropic) material was defined for concrete, and the elements were modeled in LUSAS software.

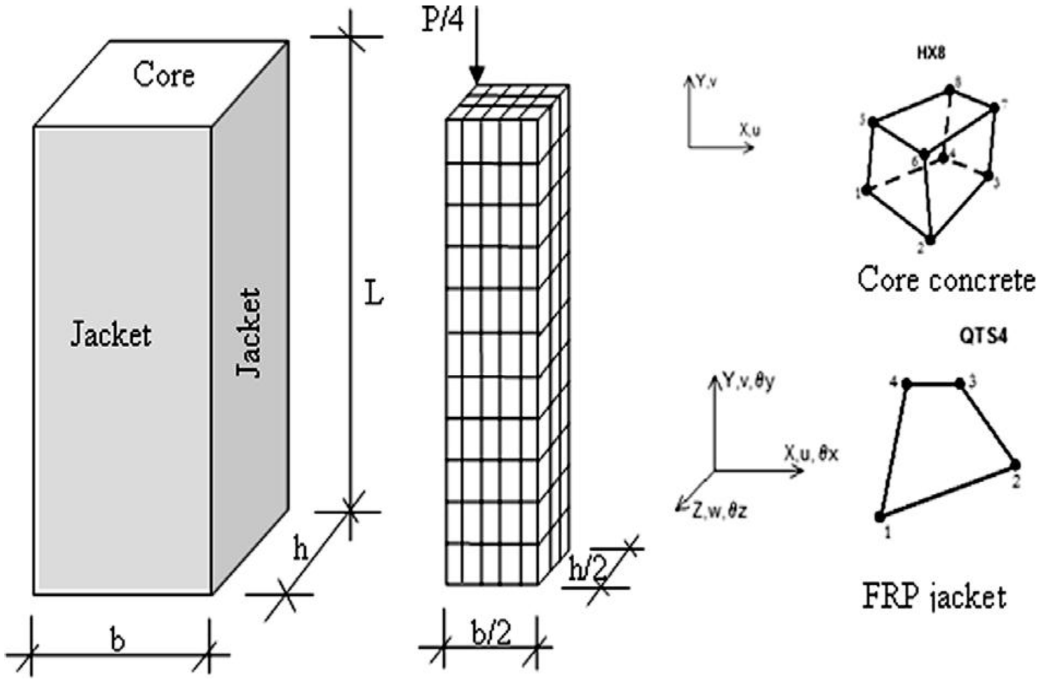
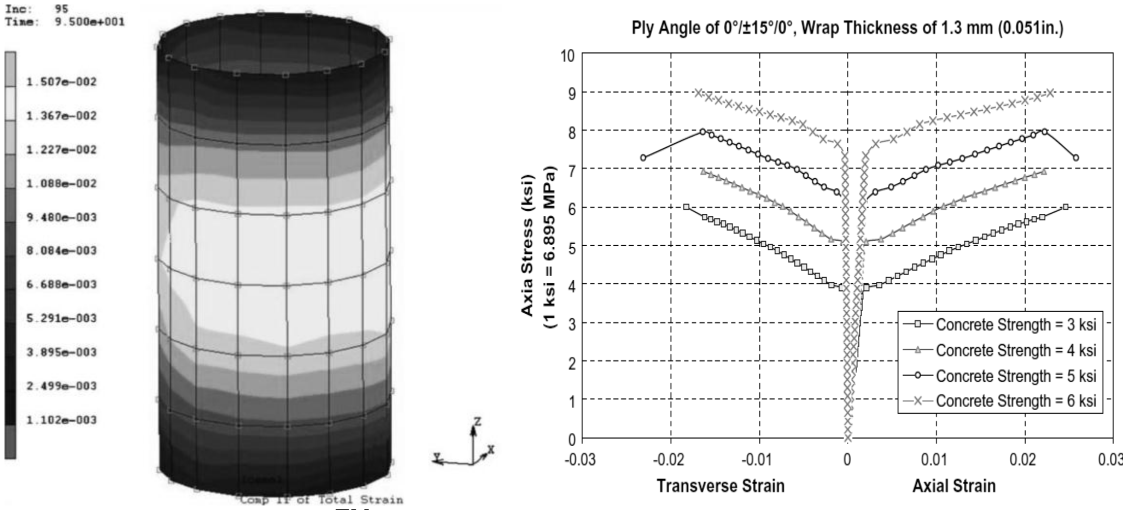


Figure 2.4: Modelling the axial force on confined column using FRP wrapping in finite element (Koksal et al., 2009)

The axial behaviour of confined concrete was simulated by using LSDYNA finite element software (Wu et al., 2009). The result was validated by comparing the analysis and experiment results (Wu et al., 2009).

Confined concrete specimens were modeled by using ANSYS. The ANSYS software can automatically generate the mesh for various geometric shapes and material properties. Results show that the response of this software is sufficient considering some coefficients, such as the dilatancy angle of FRP-confined concrete, are proposed to be zero (Mirmiran et al., 2000). The dilatancy angle (Ψ) is a measure of contractive or dilative nature of the volumetric response due to shearing. Montoya et al. (2004) introduced and employed a specific program to predict the behaviour of confined concrete axial stress and strain. New constitutive models for confined concrete that focus on the analysis of axisymmetric solids were formulated and used. Results have been found to be acceptable following the proposed modeling process of Montoya et al. (2004). Parvin and Jamwal (2006) attempted to determine the behaviour of confined concrete section by using MSCMarc™ 2001 (MSC, 2001) software, as shown in Figure 2.5.



a) Model of MSCMarc™ 2001 b) Analysis result

Figure 2.5: Modelling and result of Parvin and Jamwal (2006) for confined column

Parvin and Jamwal (2006) investigated the angle of FRP wrap on a confined section. FRP laminates in 0° show the highest axial stress-carrying capacity of confined concrete. However, given that the best angle to reach the highest axial stress is 0° for FRP laminates, ±15° has shown to be the best angle to achieve the highest strain.

Shahawy et al. (2000) studied another ANSYS finite element model of uniaxial concrete specimen. A total of 45 carbon-wrapped concrete specimen of two types of normal- and high-strength concrete with different number of FRP plies have been verified. The models contained the concrete originally developed for concrete-filled glass FRP tubes. Numerical computation of confined concrete following non-associate Drucker–Prager procedure has been conducted. The accuracy of the model