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Serra Cimilli Erkmén

AUTEUR DE LA THÈSE / AUTHOR OF THESIS

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TITRE DE LA THÈSE / TITLE OF THESIS

Murat Saatcioglu

DIRECTEUR (DIRECTRICE) DE LA THÈSE / THESIS SUPERVISOR

CO-DIRECTEUR (CO-DIRECTRICE) DE LA THÈSE / THESIS CO-SUPERVISOR

EXAMINATEURS (EXAMINATRICES) DE LA THÈSE / THESIS EXAMINERS

Brahim Benmokrane

Nove Naumoski

David Lau

Dan Palermo

Gary W. Slater

Le Doyen de la Faculté des études supérieures et postdoctorales / Dean of the Faculty of Graduate and Postdoctoral Studies

ANALYSIS AND DESIGN OF EARTHQUAKE RESISTANT FRP REINFORCED CONCRETE BUILDINGS

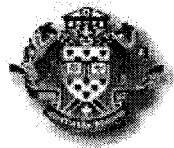
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To *Mom* and *Dad*
They were always there.

To my dear husband

Emre

He was with me every step of the way.
There are no words to describe my gratitude for his inspiration,
encouragement and love.

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Abstract

The use of fiber reinforced polymers (FRP) as construction materials is gaining acceptance in the construction industry. The primary reason for this increase is the superior performance of FRP reinforcement in corrosive environments, its long term durability, high tensile strength-to-weight ratio, electromagnetic neutrality and resistance to chemical attacks.

The use of FRP bars as concrete reinforcement is relatively new, with very few applications in practice, although externally applied FRP sheets, strips and bars for rehabilitation and seismic retrofit purposes is not uncommon. There is lack of research in performance and design of new FRP reinforced concrete structures; particularly for seismically active regions. The use of FRP bars as reinforcement is a new concept with limited experimental and analytical information. Indeed, FRP reinforced concrete structures may be lacking the required ductility for which the majority of conventional steel reinforced concrete structures are designed so that they can dissipate seismic induced energy in the event of a strong earthquake. Because of these shortcomings, the current codes and standards, including the CSA S608-02 (2002), ISIS Manual (2001) and ACI 440 (2006), have severe limitations for structural use of internally placed FRP reinforcement, especially for seismically active regions. Therefore, the objective of the current investigation was selected to establish seismic force and deformation demands for FRP reinforced concrete buildings in Canada and to develop a design procedure for earthquake resistant FRP reinforced buildings.

The scope of the research program includes six tasks; i) the development of a computer program (SEQUAKE) for static and dynamic inelastic response history analysis of FRP and steel reinforced concrete structures, incorporating hysteretic behaviour of steel and FRP reinforced concrete elements, ii) selection and design of concrete frame buildings with different heights, located in Eastern and Western Canada, reinforced with FRP and

steel rebars, iii) nonlinear dynamic analyses of selected buildings under synthetically generated earthquake records, compatible with the Uniform Hazard Spectra specified in the National Building Code of Canada (NBCC-2005) iv) review and assessment of analyses results to establish seismic force and deformation demands for FRP reinforced concrete buildings in Canada, v) review of available experimental data to establish inelastic force and deformation capacities of FRP reinforced concrete frame elements, and vi) development of seismic design guidelines for FRP reinforced concrete frame buildings in Canada.

The analysis results indicate that inelastic drift demands of FRP reinforced concrete frame buildings are similar to those observed in comparable steel reinforced concrete buildings. It is also determined that FRP reinforced concrete element should be designed to be over-reinforced to prevent tension failure of reinforcement. Inelasticity in these buildings can be achieved through the confinement of compression concrete by properly designed FRP transverse reinforcement. These findings led to a design procedure that allows a reduction in elastic design force levels. It was also established that the seismic design forces specified in NBCC 2005 for steel reinforced concrete frame buildings can be used to design FRP reinforced concrete frame buildings with appropriate modifications as explained in this thesis. Both the new computer program and the design procedure developed provide much needed tools to expand the use of internally placed FRP reinforcement in seismically active regions.

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List of Symbols

A	cross-sectional area of structural element
A_g	gross area of reinforced concrete member
A_f, A_{FRP}	cross-sectional area of FRP reinforcement
A_s	shear-reduced cross section of beam
b	width or flange width of beam
b_c	column core dimension measured centre-to-centre of perimeter hoop
b_w	web width of beam
C	internal sectional forces in compression
c	distance from the extreme compression fibre to neutral axis
$[C]$	structural damping matrix
d_b	nominal diameter of circular reinforcement
E	modulus of elasticity of structural element
E_f, E_{frp}	modulus of elasticity of FRP reinforcement
E_{fc}	compressive elastic modulus of FRP reinforcement
$\{F_t\}$	internal force vector
f'_c	specified concrete compressive strength
f_{frp}	tensile strength in FRP reinforcement
f_{frpu}	ultimate tensile strength in FRP reinforcement
G	modulus of shear of beam
h	height of structure, dimension of square column
h_s	storey height
I	moment of inertia
$[K]$	structural stiffness matrix
$[k]$	element stiffness matrix
$k_{11} \dots k_{66}$	stiffness coefficients
$k_{el,fl}$	elastic spring stiffness
$k_{in,fl}$	inelastic spring stiffness
L	element length
l_d	development length of reinforcement
M	bending moment
$[M]$	structural mass matrix

M_{cr}	cracking moment
M_f	moment due to factored loads
M_r	moment resistance
M_v	higher mode effects on base shear
M_y	yield moment
P_o	member concentric capacity in compression
p	strain hardening ratio of spring
R_d	ductility reduction factors
R_o	overstrength reduction factor
$\{R\}, \{R_t\}$	external load vector
r	strain hardening ratio of real beam
$S(T_a)$	design spectral acceleration for fundamental period T_a
s	spacing of reinforcement
T	internal sectional forces in tension
T	tensile force in reinforcement
T_a, T	fundamental period of structure
t	time in time history analysis
$\{U\}, \{U_t\}$	nodal displacement vector
$\{\dot{U}_t\}$	nodal velocity vector
$\{\ddot{U}_t\}$	nodal acceleration vector
V_d	inelastic design base shear
V_e	elastic base shear
V_f	effects of factored load
V_r	factored shear resistance
W	total weight of structure
α_l, β_l	stress-block factors
$\{\Delta F_t\}$	incremental internal force vector
Δ_{as}	member end displacement due to anchorage slip
Δ_f	member end displacement due to flexure
ΔM_t	total moment increment at each node
$\{\Delta U_t\}$	incremental displacement vector
$\{\Delta \dot{U}_t\}$	incremental velocity vector
$\{\Delta \ddot{U}_t\}$	incremental acceleration vector
Δt	time interval in time history analysis

Δ_t	member end total displacement
δ	drift ratio
δ_{ext}	extension of FRP reinforcement
ε_o	strain at peak compressive stress f'_c
ε_c	strain in concrete in compression
ε_{cu}	ultimate strain in concrete in compression
ε_{frp}	strain in FRP reinforcement in tension
ε_{frpu}	ultimate strain in FRP reinforcement in extreme layer
ξ	ratio of two end moments
ξ_i, ξ_j	critical damping ratio
λ	concrete density coefficient
ϕ_c	material strength reduction factor for concrete
ϕ_{frp}	material strength reduction factor for FRP reinforcement
ρ_{frp}	FRP reinforcement ratio
θ_{as}	member end rotation due to anchorage slip
θ, θ_{sp}	spring rotation
θ_{ch}	chord rotation
θ_f	rotation of a member due to flexure

Chapter 1

Introduction

The use of fiber reinforced polymers (FRP) as a construction material has increased in recent years. The primary reason for this increase is the non-corrosive nature of FRP and its long term durability. Other factors that contribute to increased acceptance of the material include high tensile strength-to-weight ratio, electromagnetic neutrality and resistance to chemical attack. The majority of FRP applications in building and bridge infrastructure have been in the form of sheets, either surface bonded on structural and non-structural elements or wrapped around concrete columns for the purpose of strengthening, rehabilitation and/or seismic retrofitting. Hence, the current applications in large part are intended to upgrade existing sub-standard or deficient structures. The use of FRPs in new construction is relatively new, with very few applications in practice. Particularly, the use of FRP bars as reinforcement in new concrete structures is a new concept with limited experimental and analytical information. There is lack of research in both performance and design of FRP reinforced concrete structures, particularly for seismically active regions where inelastic deformability gains a new dimension. Indeed, FRP reinforced concrete structures may be lacking the required ductility for which the majority of conventional steel reinforced concrete structures are designed so that they can dissipate seismic induced energy in the event of a strong earthquake. The proposed study aims at investigating the seismic response of FRP reinforced concrete structures while developing appropriate seismic design and detailing requirements.

1.1 General properties of FRP

The earliest use of FRP materials dates back to post-World War II era with applications in the petro-chemical industry. In the 1960s and 1970s they were mainly used in aerospace and defense industries because of their high strengths and low weights, in spite

of their high cost. By late 1980s and early 1990s the increased volume of production of fiber materials resulted in reduced costs which led to the acceptance of the material in the construction industry. The superior performance of FRPs in corrosive and magnetic environments, better strength-to-weight ratios, favorable fatigue strength, and low relaxation characteristics also contributed to the easy acceptance of the material (Bakis, et. al. 2002).

FRP composites are formed by embedding continuous fibers in a resin matrix, which binds the fibers together. Carbon, glass and aramid constitute the most common forms of fibers, while epoxy, polyester and vinylester resins are commonly used as resins. FRP composites are classified depending on the fiber type used in the matrix as carbon fiber reinforced polymer (CFRP), glass fiber reinforced polymer (GFRP) and aramid fiber reinforced polymer (AFRP) (Sonobe 1997). The amount of fibres used in a given quantity of resin is expressed as the volume fraction of fibers. This ratio ranges from about 50 to 65% in typical applications for the construction industry. The diameters of commonly used fibers range from about 6 microns to about 15 microns (Uomoto et. al. 2002).

The FRP composites, in the form of sheets, laminates, and NMSR (Near Surface Mounted Strengthening) systems are commonly used for flexural and shear strengthening of beams, flexural strengthening of slabs, flexural and shear strengthening of axially and eccentrically loaded columns and seismic retrofitting of columns. The laminates are mostly suitable for flat surfaces of beams, walls and slabs. The sheets are usually used for seismic retrofitting and strengthening of slabs and walls with openings. They are also suitable for curved structures, such as silos. NMSR systems are usually preferable when there are irregularities on concrete surface and when the strengthening system needs to be protected, for example from a possible impact, (Sonobe 1997).

Typical FRP reinforcement products for use in concrete are grids, bars, fabrics and ropes. The bars have various types of cross-sectional shapes (square, round, solid and hollow) and surface deformation patterns (ribbed, sand-coated, wrapped and sand-coated). Typical FRP rebars and grids are shown in Fig. 1.1. Tensile strength of CFRP, GFRP and

AFRP rebars range between 600 to 3690 MPa, 483 to 1600 MPa and 1720 to 2540 MPa, respectively. Their elastic modulus varies between 120 and 580 GPa for CFRP rebars, 35 and 51 GPa for GFRP rebars, 41 and 125 GPa for AFRP rebars. Figure 1.2 provides the comparison of stress-strain relationships for FRP and steel rebars. The rupturing strain of CFRP rebars ranges between 0.5 to 1.7 %, while it varies between 1.2 to 3.1 % for GFRP rebars and 1.9 to 4.4 % for AFRP rebars. Mass densities of FRP rebars vary between 1.5 and 1.6 g/cm³ for CFRP; 1.25 and 2.1 g/cm³ for GFRP; and 1.25 and 1.4 g/cm³ for AFRP rebars, respectively. The compressive strength of FRP rebars is approximately 10-50% of their tensile strength (Kawaguchi 1993). Although the AFRP rebars seem to be the most attractive among the three types of FRP rebars mentioned, they are also the most expensive, which makes CFRP and GFRP rebars more preferable in North America.

1.2 Research needs and motivation

There are uncertainties about the feasibility and effectiveness of the use of FRP reinforcement in new concrete construction. FRP does not exhibit inelastic behaviour, developing brittle material failure upon attaining its elastic limit. Though some research is underway to develop hybrid bars with limited inelasticity, these efforts have not resulted in the development of reliable FRP reinforcing bars with similar characteristics as conventional steel reinforcement. The elongation capacity of FRP material is usually small, limiting the deformability of FRP reinforced concrete elements. Their compression capacity is significantly less than their tensile strengths. Because of these shortcomings, the current codes and standards, including the Canadian Standards Association's S608-02 (2002), Canadian Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS) Design Recommendations (2001), and the American Concrete Institute's ACI-440 (2006) limit the use of FRP reinforcement in structural concrete. These documents often limit the use of FRP reinforcement to flexural and shear reinforcement for use in beams and slabs, hindering widespread applications to bridge and building infrastructure.

Ductility and inelastic deformability are important characteristics that are expected from earthquake resistant elements for the dissipation of seismic induced energy. Research is

badly needed for developing design and detailing guidelines for FRP reinforced concrete structures. Research is also needed to establish structural response of FRP reinforced concrete structures to dynamic excitations. Furthermore, the computation of seismic response generally requires complex analytical models, especially to simulate the hysteretic behaviour of elements within the inelastic range of deformations. While several computer programs have been developed for nonlinear dynamic analyses of steel reinforced concrete structures, there is no such tool available for the dynamic inelastic response history analysis of FRP reinforced concrete structures. The aim of the proposed study is to fulfill these needs by developing non-linear dynamic analysis computer software, investigating seismic response of FRP reinforced concrete structures and developing design guidelines and design aids.

1.3 Previous research

Literature review was conducted on the use of FRP as concrete reinforcement in new concrete structures. The previous work has been classified and presented under five main groups; i) research on FRP reinforced members under monotonically increasing axial load and bending, ii) research on FRP reinforced concrete columns and beams under reversed cyclic loading, iii) research on ductility and moment redistribution, iv) analytical models developed for the behavior of FRP reinforced concrete members, and v) design procedures developed for FRP reinforced concrete structures.

1.3.1 FRP reinforced members under combined axial load and bending

Kawaguchi (1993) investigated the capacity and performance of FRP reinforced concrete members under combined axial load and bending, using AFRP as main reinforcement. The specimens were subjected to monotonically increasing eccentric tension or compression. The specimens were designed such that in both the eccentric tension and compression tests the failure of all specimens was caused by the crushing of concrete under compression, without the rupturing of FRP reinforcement. The researcher concluded that the ultimate strength of FRP members under axial load and bending may

be evaluated by the conventional beam theory so long as certain minimum amount of bond is maintained between the concrete and the FRP bars.

Daniali and Paramanantham (1994) investigated the behavior of FRP reinforced concrete columns using GFRP as longitudinal reinforcement. They presented the following expression for the concentric capacity of columns, assuming that an axially loaded column fails when the entire cross-section reaches the critical strain of 0.003.

$$P_o = 0.85f'_c (A_g - A_f) + 0.003E_f A_f \quad (1.1)$$

Where A_g is the gross area of concrete, A_f is the total cross-sectional area of longitudinal FRP reinforcement and E_f is the modulus of elasticity of the FRP rebar. Also, they presented two sets of equations in order to plot interaction diagrams for FRP reinforced concrete column. The first set assumes a concrete compressive strain of 0.003 and the computation of strains in FRP bars from strain compatibility. The second set assumes FRP reinforcement strain equal to the yield strain of steel reinforcement, based on the argument that “in order to provide the same serviceability such as crack width and lateral deformation, FRP reinforced concrete columns should have the same stiffness as identical columns reinforced with steel”. Consequently, the suggested expressions provide interaction diagram similar to those for steel reinforced concrete columns.

Alsayed et. al. (1999) tested 15 columns in order to investigate the influence of replacing some or all of steel bars in columns by an equal volume of GFRP bars. The columns were subjected to concentric monotonic axial loading. The results indicated that replacing longitudinal steel bars by GFRP bars reduced the axial capacity of columns by 13%. The results also showed that replacing steel ties by GFRP ties reduced the axial capacity of columns by 10 %. They showed that the analytical expression commonly used for computing concentric capacities of steel reinforced columns (the product of the sectional area of the column and 85 percent of concrete strength and adding the product of steel strength and steel sectional area) can also be used for FRP reinforced concrete columns, after applying a reduction factor of 0.6 on the strength of FRP bars.

Sharbatdar (2003) tested FRP reinforced columns under eccentric loading and FRP reinforced concrete cylinders under concentric loading. He used CFRP longitudinal rebars as main reinforcement and CFRP grids as transverse reinforcement. The test results showed that the columns were able to develop their moment capacities as governed by concrete crushing. He also observed that CFRP reinforcement used as column compression reinforcement maintained its load resistance until after the crushing of the surrounding concrete occurred.

1.3.2 FRP reinforced columns and beams under cycling loading

Kobayashi and Fujisaki (1995) carried out compression tests on FRP reinforced concrete columns under cyclic loading in order to investigate the influence of cyclic loading on the compression strength of FRP reinforcement. They used AFRP, CFRP and GFRP bars as longitudinal reinforcements. They observed that the compressive capacity of the AFRP, GFRP and CFRP was approximately 10%, 30-40% and 30-50% of their tensile capacity, respectively. They also reported that there was a 20 -50 % reduction of the compressive capacity for the AFRP and GFRP reinforcements under cyclic loading while CFRP reinforcement was not affected by load reversals. It was also suggested that the concentric capacity of FRP reinforced concrete columns can be computed as the product of the sectional area of the column and 85 percent of concrete strength, neglecting the contribution of FRP bars.

Saatcioglu and Sharbatdar (2000) tested full-size steel reinforced columns with CFRP grids as transverse reinforcement under cyclic loading. They reported that FRP grids can be used as column confinement for improved seismic performance.

Sharbatdar (2003) tested FRP reinforced concrete columns and beams under reversed cyclic loading in order to investigate their seismic behavior. CFRP bars and grids were used as longitudinal and transverse reinforcements. Both columns and beams were able to develop their expected moment capacities. Failure in most columns under high axial compression and reverse cyclic loading was observed in the form of spalling of concrete cover, followed by the buckling of FRP bars in compression and subsequent crushing of

concrete. Beams without axial compression failed by rupturing of tension reinforcement. However, it was also stated that concrete crushing can be obtained at ultimate prior to the tension failure of FRP provided that the concrete is confined sufficiently to prevent brittle failure. It was also observed that FRP reinforced columns developed limited drift capacities, slightly higher than the limits specified in the National Building Code of Canada. Following expression was used for the concentric capacity of FRP reinforced concrete columns, assuming that the failure under concentric compression occurred at 0.002 strain:

$$P_o = 0.85f'_c (A_g - A'_{FRP}) + 0.002E_{fc}A_{FRP} \quad (1.2)$$

Where; A_{FRP} and E_{fc} are the total core area and compressive elastic modulus of longitudinal FRP bars. A'_{FRP} is the total area of sand-coated FRP reinforcement, including the area of sand coating.

1.3.3 Ductility and moment redistribution

Nakano et. al. (1993) carried out an experimental investigation in order to study the flexural performance of FRP reinforced concrete beams. They concluded that the conventional methods used in concrete beams reinforced with steel bars could also be used for the evaluation of flexural performance of concrete beams reinforced with FRP bars. Also, their results showed that even though the FRP bars do not exhibit yielding, the gradual compressive failure of concrete can provide adequate ductility in over-reinforced elements.

Taniguchi et. al. (1993) investigated the ductility of reinforced-concrete beams when the concrete is confined by CFRP transverse spiral reinforcement. They concluded that it was possible to improve the ductility of members using FRP as transverse reinforcement provided that sufficiently small pitch was used. They also stated that it was possible to provide FRP reinforced concrete members with properties similar to steel reinforced members.

Morais-Burgoyne (2001) studied the energy absorption capacity of steel and FRP prestressed concrete sections experimentally. Two types of concrete were used, confined either with; i) conventional steel spirals or ii) AFRP spirals. They concluded that when the AFRP spirals were used, the plastic energy absorbed in over-reinforced sections matched with that of under-reinforced sections with steel reinforcement.

Mostofinejad (1997) tested continuous two-span beams with over and under-reinforced sections having CFRP and steel reinforcement to evaluate their ductility and moment redistribution capacities. The compression zones of some of the over-reinforced beams were confined by CFRP grids. It was concluded that concrete nonlinearity had a significant effect on the ductility of continuous beams, enabling moment redistribution in FRP reinforced beams. The study showed that approximately 50% of the available redistribution potential of a conventional steel concrete beam was achieved in FRP reinforced concrete beams.

Orozco and Maji (2004) tested concrete beams reinforced with steel and CFRP bars to compare ductility capacities of these two types of beams. It was observed that the ductility of FRP reinforced beams was less than that of steel reinforced beams. However, it was found that energy dissipation due to concrete cracking ensured sufficient ductility in the FRP reinforced beams.

Fukuyama et. al. (1995) tested a three-storey, half scale FRP reinforced concrete frame subjected to reversed cyclic loading in order to investigate its structural performance. They concluded that designing a concrete frame using FRP bars as main and shear reinforcement was feasible. The researchers also indicated that it was possible to investigate the load-deformation relationship of a frame through nonlinear analysis by properly evaluating decreases in member stiffness caused by concrete crushing under inelastic deformation reversals.

1.3.4 Analytical models

Mostofinejad (1997) proposed a bi-linear model for the moment-curvature response of FRP reinforced concrete beams, which is illustrated in Fig. 1.3. In the figure, points A and D define the moments and curvatures at cracking and ultimate, respectively. Point C is located where moment is equal to 80 percent of the ultimate moment capacity and point B is located on line OC where moment is equal to the cracking moment. It was suggested that the deviation of broken line OAB from the straight line OB could be neglected, since its effect may not be significant on deflection calculations. The resulting curve OBCD gives the bi-linear model proposed.

Sharbatdar and Saatcioglu (2003) developed an analytical model for the hysteretic moment-displacement relationship of FRP reinforced concrete members. The model was adopted and used in developing dynamic analysis computer software in the current investigation. The primary curve and hysteretic rules are explained in detail in Section 2.2.3.4.

1.3.5 Design recommendations

1.3.5.1 Flexural capacity

Previous research shows that the flexural capacity of FRP reinforced concrete members can be calculated using many of the same assumptions used for steel reinforced concrete members. The CSA Standard S806-02 (2002), the ISIS Design Recommendations (2001), the Japanese Design Guideline for FRP Reinforced Concrete (JSCE1997) and the ACI Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars (ACI 440.1R 2006) recommend the following assumptions for the computation of sectional forces within a cross section:

- Strain in FRP reinforcement and concrete are proportional to the distance from neutral axis in cases where there is perfect bond.
- Tensile strength of concrete is ignored.
- Linear tensile stress-strain relationship of FRP reinforcements is assumed up to failure.

- Compressive resistance of FRP reinforcement is ignored.

1.3.5.2 Failure modes

Codes and design guidelines define three failure modes:

- *Balanced Failure:* Rupture of FRP reinforcement and crushing of concrete occur simultaneously.
- *Compression Failure:* Concrete crushing occurs while the FRP reinforcement remains in the elastic range.
- *Tension Failure:* FRP reinforcement ruptures prior to the crushing of concrete.

Rectangular stress block assumption can be used when the member fails in compression. Ultimate strain at the extreme concrete compression fibre is recommended to be 0.0035 by CSA S806-02 (2002) and ISIS (2001). Accordingly, a concrete stress of $\alpha_1 \phi_c f'_c$ is assumed to be uniformly distributed as a rectangular stress block within a distance $a = \beta_1 c$, measured from the maximum compressive fiber, where c is distance from the extreme compression fibre to neutral axis, ϕ_c is material strength reduction factor which is taken as 0.65 and f'_c is the specified concrete compressive strength. The coefficients α_1 and β_1 can be calculated as follows;

$$\alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67 \quad (1.3)$$

$$\beta_1 = 0.97 - 0.0025 f'_c \geq 0.67 \quad (1.4)$$

When the crushing of concrete occurs before FRP rupture, the stress in FRP f_{frp} is smaller than its tensile strength. The ISIS design manual (2001) provides the following expression for f_{frp} :

$$f_{frp} = 0.5 E_{frp} \epsilon_{cu} \left[\left(1 + \frac{4 \alpha_1 \beta_1 f'_c}{\rho_{frp} E_{frp} \epsilon_{cu}} \right)^{1/2} - 1 \right] \quad (1.5)$$

Where; ρ_{frp} is the reinforcement ratio which can be calculated as:

$$\rho_{frp} = \frac{A_{frp}}{bd} \quad (1.6)$$

In the case of tension failure, the rectangular stress distribution is no longer valid and the strain compability analysis is required in order to calculate the concrete compressive strain ε_c , using the ultimate tensile strain of FRP reinforcement and an assumed value of neutral axis depth c . The iterative process is repeated until the following equilibrium is satisfied:

$$C = T \quad (1.7)$$

Where; C and T are the internal sectional forces in compressive and tension. The ISIS design manual (2001) defines these forces for sections with single layer of reinforcement as follows;

$$C = \alpha \phi_c f'_c \beta c b \quad (1.8)$$

$$T = A_{frp} \phi_{frp} \varepsilon_{frp} E_{frp} \quad (1.9)$$

The same manual suggests tables and figures in order to determine the stress-block factors α and β for concrete compressive strains for ε_c values of up to 0.0035. When the concrete compressive strain ε_c is equal to 0.0035, the values of α and β are identical to coefficients α_l and β_l . In Eq. (1.9), A_{frp} is the area of FRP reinforcement; ϕ_{frp} is the FRP strength reduction factor, which is equal to 0.8. ε_{frp} is the strain in FRP and E_{frp} is the modulus of elasticity of FRP.

The manual also gives the following expression for balanced reinforcement ratio:

$$\rho_{frp} = \alpha_1 \beta_1 \frac{f'_c}{f_{frpu}} \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{frpu}} \right) \quad (1.10)$$

Here; f_{frpu} is the ultimate tensile strength of FRP, ε_{cu} is the ultimate strain in concrete in compression equal to 0.0035, ε_{frpu} is the ultimate strain in FRP bar in extreme layer.

In cases where there is more than one layer of tension reinforcement, the failure of the outermost layer controls the section failure in tension.

1.3.5.3 Design philosophy

CSA S806-02 (2002) requires that all FRP reinforced concrete sections are designed such that the section failure is initiated by the crushing of concrete in compression. On the other hand, ISIS (2001) as well as ACI 440.1R-06 (2006) and JSCE (1997) allow both compression and tension failure modes. However, when FRP rupture is allowed as the failure mode, because this mode of failure is sudden and catastrophic, the margin of safety is set at a higher value than that used for traditional steel-reinforced concrete sections. A minimum amount of flexural reinforcement is required to prevent sudden failure shortly after the cracking of concrete. ISIS (2001) requires that the moment resistance, M_r , should be at least 50 percent greater than either the cracking moment M_{cr} or the moment due to factored loads M_f if the first requirement can not be satisfied:

$$M_r \geq 1.5M_{cr} \quad (1.11)$$

$$M_r \geq 1.5M_f \quad (1.12)$$

Alternatively, the following two expressions can be used to calculate the minimum reinforcement areas for rectangular and T beams, respectively:

$$A_{frp,min} = \frac{5\sqrt{f'_c}}{12f_{frpu}} bd \quad (1.13)$$

$$A_{frp,min} = \frac{5\sqrt{f'_c}}{6f_{frpu}} b_w d \quad (1.14)$$

The minimum reinforcement area given by ACI 440.1R-06 (2006) is as follows:

$$A_{frp,min} = \frac{4.9\sqrt{f'_c}}{f_{fu}} b_w d \geq \frac{330}{f_{fu}} b_w d \quad (1.15)$$

Where; f_{fu} is the design tensile strength of FRP. According to JSCE (1997), the minimum tensile reinforcement area is computed as the higher of the following two quantities:

$$A_{frp,min} = 35k_1 \frac{f_{tk}}{f_{fuk}} b_w d \quad (1.16)$$

$$A_{frp,min} = 0.02b_w d \quad (1.17)$$

Here, f_{tk} is the characteristic value of the tensile strength of concrete, f_{fuk} is the characteristic value of the tensile strength of reinforcement and k_1 is a coefficient defined below:

$$k_1 = \frac{0.6}{\sqrt[3]{h}} \quad (1.18)$$

Where; h is the total member depth, provided that $0.4 \leq k_1 \leq 1.0$.

The same design codes, standards and guidelines outlined above recommend limit states design procedures for the design of FRP reinforced concrete structures such that the building components and connections are designed to have factored resistances at least equal to the effects of factored loads. These structures are designed using the ultimate limit states principles to ensure sufficient strength and then checked for serviceability limit states, such as those for deflections and crack control. In FRP reinforced concrete structures, generally the later (serviceability) controls the design because of the low elastic modulus of FRP reinforcement.

1.3.5.4 Strength reduction factors

The strength reduction factors recommended for FRP reinforced concrete is more conservative than those used for conventional steel reinforced concrete because of the

brittle behaviour of FRP reinforcement. JSCE (1997) limits the tensile stress in FRP bars by multiplying the characteristic value of creep failure strength f_{fck} by a reduction factor of 0.8 to account for the fact that FRP bars undergo failure at lower values than their short term strength values, under sustained stresses, unlike the steel reinforcement (i.e. creep failure). The code also limits the strength of FRP reinforcement to 70% of its characteristic value in tension.

ACI 440.1R-06 (2006) gives the following equation for the strength reduction factor for flexure:

$$\phi = \begin{cases} 0.55 \text{ for } \rho_f \leq \rho_{fb} \\ 0.3 + 0.25 \frac{\rho_f}{\rho_{fb}} & \rho_f < 1.4\rho_{fb} \\ 0.65 \text{ for } \rho_f \geq 1.4\rho_{fb} \end{cases} \quad (1.19)$$

The reductions factors based on ISIS (2001) are 0.8, 0.6 and 0.4 for CFRP, AFRP and GFRP respectively. According to CSA S806-02 (2002), for non-prestressed FRP reinforcements, the reduction factor is 0.75. Also, both ISIS (2001) and CSA S806-02 (2002) recommendations for the strength reduction factor of concrete is 0.65.

1.3.5.5 Minimum thickness for design

Design codes specify span to thickness ratios to avoid excessive deflection of steel reinforced concrete beams and slabs. In ISIS (2001) the equation for span to depth ratio for FRP reinforced members is given as follows:

$$\left(\frac{l_n}{h} \right)_{frp} = \left(\frac{l_n}{h} \right)_s \left(\frac{\epsilon_s}{\epsilon_{frp}} \right)^{\alpha_d} \quad (1.20)$$

In the above expression, l_n is the member length, h is the member thickness, ϵ is the maximum strain allowed in reinforcement under service load conditions and α_d is a dimensionless coefficient, which is taken as 0.5 for rectangular sections and

$0.5+0.03(b/b_w)$ for T-sections, where b and b_w are flange and web widths, respectively. ACI 440.1R-06 (2006) minimum depth requirements are similar to those specified by CSA Standard A23.3-04 (2004) for steel reinforced concrete beams and one-way slabs. Accordingly, the following span-to-depth ratio is specified in ACI 440.1R-06 for deflection control, below which deflection calculations are required:

$$\frac{l}{h} = \frac{48\eta}{5K_1} \left(\frac{1-k}{\epsilon_f} \right) \left(\frac{\Delta}{l} \right)_{\max} \quad (1.21)$$

In Eq. (1.21), $\eta=d/h$, $(\Delta/l)_{\max}$ is the limiting service load deflection-span ratio, K_1 is a parameter that accounts for boundary conditions and it may be taken as 1.0, 0.8, 0.6 or 2.4 for uniformly loaded simply-supported, one end continuous, two ends continuous and cantilever spans, respectively. The term ϵ_f is the strain in the FRP bar under service loads, evaluated at midspan except for cantilever beams for which it will be evaluated at the support. The term k can be calculated as follows:

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad (1.22)$$

Where; ρ_f is the FRP reinforcement ratio and n_f is the modular ratio between the FRP reinforcement and the concrete. Recommended minimum thickness for design of one-way slabs and beams are provided in the same code. However the code also adds that these thicknesses are for convenience in establishing member proportions for design only and recommends computing the deflections in order to compare them to acceptable limits. CSA S806-02 (2002) also recommends calculating and limiting the immediate deflections as described in the standard.

1.3.5.6 Development length

Length of an FRP reinforcing bar within which the stress can increase from zero to the ultimate strength, f_{rup} is called the development length. In CSA S806-02 (2002) the development length, l_d , of tension bars is given as:

$$l_d = 1.15 \frac{k_1 k_2 k_3 k_4 k_5}{d_{cs}} \frac{f_f}{\sqrt{f'_c}} A_b \quad (1.23)$$

Where; k_1 , k_2 , k_3 , k_4 and k_5 are factors for bar location, concrete density, bar size, fibre type and bar surface profile, respectively. The term d_{cs} is the smaller of the distance from the closest concrete surface to the center of the bar being developed or two-thirds of the centre-to-centre spacing of the bars being developed, not be taken greater than $2.5d_b$. The terms f_f and A_b are the design stress in FRP tension reinforcement at ultimate limit state and the area of individual bar, respectively. The following expression can be used for the calculation of development length, provided that the clear cover and clear spacing of bars being developed are at least $1.5d_b$ and $1.8d_b$ respectively, where d_b is the nominal diameter of a circular bar or equivalent of a rectangular bar.

$$l_d = 0.5 k_1 k_2 k_3 k_4 k_5 \frac{f_f}{\sqrt{f'_c}} d_b \quad (1.24)$$

ISIS design manual (2001) refers to the development length requirements specified in the ACI 440 Report (1999). Therefore, the updated recommendations of ACI 440.1R-06 (2006) are summarized below as the updated ACI design guidelines for development length. Accordingly, the stress in a straight bar, f_{fe} , at the end of the embedment length, l_e , is given as follows, in SI units:

$$f_{fe} = \frac{0.083 \sqrt{f'_c}}{\alpha} \left(13.6 \frac{l_e}{d_b} + \frac{C}{d_b} \frac{l_e}{d_b} + 340 \right) \leq f_{fu} \quad (1.25)$$

In the above expression, C is the cover dimension, d_b is the diameter of reinforcing bar and C/d_b is limited to 3.5. The term α is a factor accounting for bar location. Also, a strength reduction factor of 0.55 is recommended for flexure when the mode of failure is bond.

JSCE (1997) recommends calculating the basic development length of reinforcement types which undergo bond splitting failure as follows, provided that $l_d > 20\phi$:

$$l_d = \alpha_1 \frac{f_d}{4f_{bod}} \phi \quad (1.26)$$

Where; ϕ is the diameter of the main reinforcement and α_1 is a factor calculated based on the cover concrete, distance between the bars, diameter of the main reinforcement and the area of transverse reinforcement. The term f_d is the design tensile strength of FRP bar, f_{bod} is the design bond strength of concrete which is calculated according to Eq. (1.27):

$$f_{bod} = 0.28\alpha_2 \frac{f'_{ck}{}^{2/3}}{\gamma_c} (N/mm^2) \quad (1.27)$$

In the above expression, f_{bod} is limited to 3.2 N/mm^2 . The term α_2 is the modification factor for bond stress of FRP bar, f'_{ck} is the characteristic compressive strength of concrete and γ_c is the material safety factor for concrete.

1.3.5.7 Shear capacity

CSA S806-02 (2002) recommends that members subjected to shear will be proportioned such that the factored shear resistance, V_r , of FRP reinforced concrete member is larger than the effects of factored load, V_f . The calculation of factored shear resistance of FRP transverse reinforcement is given as:

$$V_r = V_c + V_{sf} \leq V_c + 0.6\lambda\phi_c\sqrt{f'_c}b_wd \quad (1.28)$$

Here; the factor λ reflects the effect of concrete density. The shear resistance of concrete for sections having at least minimum transverse shear reinforcement or sections with an effective depth not exceeding 300 mm is given below:

$$0.1\lambda\phi_c\sqrt{f'_c}b_wd \leq V_c = 0.035\lambda\phi_c\left(f'_c\rho_wE_F\frac{V_f}{M_f}d\right)^{1/3}b_wd \leq 0.2\lambda\phi_c\sqrt{f'_c}b_wd \quad (1.29)$$

The quantity $\frac{V_f d}{M_f}$ is limited to 1.0. This ratio is computed as the value at the section under consideration corresponding to the load combination causing maximum moment to occur at the section. The shear resistance of concrete for sections not having sufficient amount of transverse reinforcement and with effective depths exceeding 300 mm can be calculated as follows:

$$V_c = \left(\frac{130}{1000 + d}\right)\lambda\phi_c\sqrt{f'_c}b_wd \geq 0.08\lambda\phi_c\sqrt{f'_c}b_wd \quad (1.30)$$

The shear resistance of the FRP transverse reinforcements for beams with FRP flexural and shear reinforcement are calculated from:

$$V_{sF} = \frac{0.4\phi_F A_v f_{Fu} d}{s} \quad (1.31)$$

In the above expression, s and f_{Fu} represent the spacing and ultimate strength of shear reinforcement. In all regions of flexural members where the factored shear force, V_F , is larger than $0.5V_c$ (exceptions apply for some beams, slabs and footings), a minimum area of shear reinforcement is required to be provided. The following provides the minimum area of shear reinforcement:

$$A_v = \frac{0.3\sqrt{f'_c}b_ws}{f_{Fh}} \quad (1.32)$$

The term f_{Fh} is the design stress of shear reinforcement. ISIS (2001) recommends a similar design approach based on the simplified method of CSAA23.3-94(1994). Accordingly, the maximum strain in FRP stirrups at service loads is limited to 0.002 to control shear crack width in beams.

ACI 440.1R-06 (2006) recommends calculating the shear resistance of concrete for SI units as follows:

$$V_c = 5k\sqrt{f'_c}b_wd \quad (1.33)$$

Eq. (1.33) accounts for the axial stiffness of FRP reinforcement through the depth of neutral axis. Coefficient k can be calculated as in Eq. (1.22). The shear resistance of FRP stirrups is given as:

$$V_f = \frac{A_{fv}f_{fv}d}{s} \quad (1.34)$$

ACI 440.1R-06 (2006) limits the stress in FRP shear reinforcement to $f_{fv} = 0.004E_f \leq f_{fb}$ in order to control shear cracks where; f_{fv} is the tensile strength of FRP for shear design and f_{fb} is the strength of the bent portion of FRP bar. The required spacing and the shear reinforcement area can be computed from Eq. (1.35) when the reinforcement is perpendicular to the axis of member.

$$\frac{A_{fv}}{s} = \frac{(V_u - \phi V_c)}{\phi f_{fv}d} \quad (1.35)$$

Here; A_{fv} is the amount of shear reinforcement within spacing s , and V_u is the factored shear force acting at the section. ACI 440.1R-06 (2006) recommends a minimum amount of shear reinforcement when V_u exceeds $\phi V_c/2$ as follows, for SI units:

$$A_{fv,\min} = 0.35 \frac{b_ws}{f_{fv}} \quad (1.36)$$

1.3.5.8 Members under flexure and axial load

ACI 440.1R-06 (2006) and ISIS (2001) do not include provisions for members under combined flexure and compressive axial load, while CSA S806-02 (2002) prohibits the use of FRP bars as longitudinal reinforcement in such members. In CSA S806-02 (2002), for members reinforced with longitudinal steel bars and FRP transverse reinforcements, the following requirements are listed:

- “ (a) FRP ties will consist of one or more of the following:
- preshaped rectilinear ties with corners having an angle of not more than 135°;
 - prefabricated rectilinear grids with corners having an angle of not more than 135°;
 - cross ties with hooks where the hooks engage peripheral longitudinal bars;
 - preshaped circular ties or rings.
- (b) The spacing of FRP ties should not exceed the smaller of the following dimensions:
- 16 times the diameter of the smallest longitudinal bar or the smallest bar in bundle;
 - 48 times the minimum cross-sectional dimension (or diameter) of FRP tie or grid;
 - the least dimension of the compression member;
 - 300 mm in compression members containing bundle bars.

For specified concrete compressive strength in excess of 50 MPa, the tie or grid spacing determined above shall be multiplied by 0.75.”

JSCE (1997) gives the following expression for the upper limit of axial compressive capacity N'_{oud} , when ties are used:

$$N'_{oud} = 0.85 f'_{cd} \frac{A_c}{\gamma_b} \quad (1.37)$$

Where, A_c is the cross-sectional area of concrete, f_{cd} is the design compressive strength of concrete and γ_b is the member safety factor which is generally taken as 1.3. Eq. (1.37) or the following expression is recommended for the axial capacity of columns where spiral reinforcement is used, whichever gives the larger value:

$$N'_{oud} = \frac{(0.85f'_{cd} A_e + 2.5E_{sp}\epsilon_{fspd}A_{spe})}{\gamma_b} \quad (1.38)$$

Where, A_e is the cross-sectional are of concrete enclosed by spiral reinforcement, A_{spe} is the equivalent cross-sectional area of spiral reinforcement, E_{sp} is the modulus of elasticity of spiral reinforcement, ϵ_{fspd} is the design value of strain in spiral reinforcement at yield strength, which can be generally taken as 0.002.

1.3.6 Conclusions from previous research

The following provides a summary of conclusions obtained from previous research, including the design recommendations and guidelines proposed for FRP reinforced concrete structures;

- Carbon-FRP reinforcement is more suitable than aramid and glass reinforcement for use in concrete members subjected to cyclic loading (Kobayashi and Fujisaki 1995).
- CFRP reinforcement used as column compression reinforcement can be assumed to maintain its load resistance until after the crushing of surrounding concrete was observed (Sharbatdar 2003). The conventional flexural beam theory used for concrete sections reinforced with steel bars was found to be applicable to sections reinforced with CFRP bars provided that sufficient bond can be maintained between the concrete and CFRP bars (Kawaguchi 1993 and Nakano et. al. 1993).
- The concentric capacity of FRP reinforced concrete columns can be computed by either assuming that it is equal to the product of the sectional area of column and 85 percent of concrete strength, neglecting the contribution of FRP bars (Kobayashi and Fujisaki 1995 and Alsayed et. al. 1999) or superimposing the

contribution of concrete and FRP rebar, assuming the failure under concentric compression occurs at 0.002-0.003 strain. (Sharbatdar 2003 and Daniali and Paramanantham 1994)

- Flexural failure triggered by the crushing of concrete, as opposed to the rupturing of FRP bars, may be more appropriate for earthquake resistant construction (Sharbatdar 2003, Nakano et. al. 1993, Taniguchi et. al. 1993, Morais-Burgoyne 2001, Mostofinejad 1997 and Orozco and Maji 2004).
- CFRP grids have sufficient strength and deformability to confine concrete (Saatcioglu and Sharbatdar 2000).
- Once FRP reinforcement suffers from a compression failure, it cannot maintain its tensile strength (Saatcioglu and Sharbatdar 2000).
- Inelastic response of FRP reinforced structures can be analyzed by using the hysteretic model proposed by Sharbatdar and Saatcioglu (2003).

1.4. Objective

The objective of the proposed investigation is to investigate dynamic inelastic response of FRP reinforced concrete structures under seismic loading, while developing design requirements for such structures. The objective includes the determination of seismic force and deformation demands under the Canadian seismicity defined by the 2005 edition on NBCC while also establishing strength and deformation capacities of FRP reinforced concrete buildings under combined gravity and seismic loading.

1.5. Scope

The scope of the proposed investigation consists of three main tasks, as indicated below:

- Development of a computer program for planar static and dynamic inelastic response history analysis of reinforced concrete structures. The program features include:
 - Static analysis of structures.
 - Dynamic inelastic analysis of structures, using the step-by-step time-integration method.

- Inelastic analysis by taking into account the hysteretic behaviour of steel and FRP reinforced concrete members.
- Introduction of the effect of anchorage slip of FRP reinforcement as an additional deformation components for the assessment of inelastic response of CFRP reinforced concrete members.
- Investigation of nonlinear response of FRP reinforced concrete structures and comparisons with conventional structures reinforced with steel re-bars. This task includes the following steps:
 - Selection and design of concrete frame buildings with different heights, located in Eastern and Western Canada, reinforced with CFRP and steel rebars.
 - Nonlinear dynamic analyses of selected buildings under synthetically generated earthquake records, compatible with the Uniform Hazard Spectra specified in the National Building Code of Canada (NBCC-2005).
 - Review and evaluation of analyses results to assess seismic force and deformation demands for CFRP reinforced concrete buildings in Canada.
- Development of Design Guidelines. This task includes review of available test data and design guidelines to formulate and propose design expressions for required strength and deformability of earthquake resistant FRP reinforced concrete buildings.

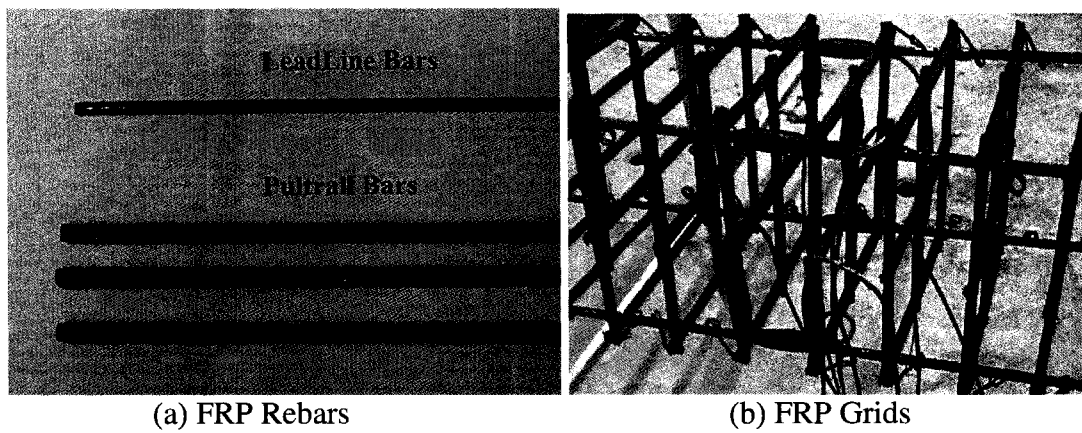


Figure 1.1 Typical FRP Rebars and Grids

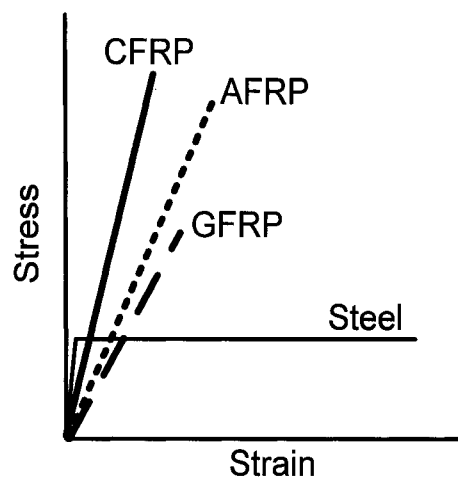


Figure 1.2 Tensile stress-strain behaviour of FRP and steel rebars

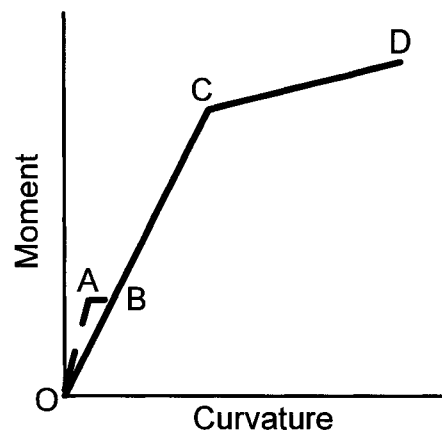


Figure 1.3 Bi-linear model of Mostofinejad (1997) for moment-curvature relationship of FRP reinforced concrete sections

Chapter 2

Development of Computer Software

SEQUAKE

2.1. Introduction

Computer program, SEQUAKE (program for Structural analysis under EarthQUAKE loading) is developed to analyze three-dimensional nonlinear static and dynamic response of reinforced concrete structures. The basic properties of the program can be summarized as follows;

- Members are idealized as beam-column elements.
- Dynamic response is determined for each time increment (time step) using time-integration method.
- Plastic deformations of elements are concentrated in plastic hinges provided at member end springs.
- The changes in the stiffness matrix of an element are taken into account by considering the tangent stiffness on the moment-rotation relationship of the springs.
- The moment-rotation relationships of end springs are obtained by incorporating hysteretic models developed for flexural behaviour of FRP and steel reinforced concrete members.
- For steel reinforced elements, the hysteretic model of Clough (1966) is adopted.
- For FRP reinforced elements, the hysteretic model of Shahbatdar and Saatcioglu (2003) is adopted.

2.2. Program structure

2.2.1 Static analysis

SEQUAKE was originally developed by Erkmen (2001) for three-dimensional static analyses of structures. The program follows the basic steps of finite element programs (Bathe 1982, Krishnamoorthy 1994). The structure is considered as an assemblage of finite number of members. Local sign convention for an element is shown in Fig.2.1.

Stiffness coefficients of the beam element are given throughout the study in the form of two-dimensional matrix elements. Similarly, the coefficients in the perpendicular direction can be easily calculated based on the steps explained in the following sections. Stiffness coefficients for a two-dimensional beam element stiffness matrix are given below in the form of a local stiffness matrix, k :

$$k = \begin{bmatrix} k_{11} & k_{12} & k_{13} & k_{14} & k_{15} & k_{16} \\ k_{21} & k_{22} & k_{23} & k_{24} & k_{25} & k_{26} \\ k_{31} & k_{32} & k_{33} & k_{34} & k_{35} & k_{36} \\ k_{41} & k_{42} & k_{43} & k_{44} & k_{45} & k_{46} \\ k_{51} & k_{52} & k_{53} & k_{54} & k_{55} & k_{56} \\ k_{61} & k_{62} & k_{63} & k_{64} & k_{65} & k_{66} \end{bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \quad (2.1)$$

E is the modulus of elasticity, A is the sectional area, I is the moment of inertia and L is the element length. The structural stiffness matrix, $[K]$, is obtained by assembling element stiffnesses through the use of Direct Stiffness Method. The assemblage procedures used in the program for all methods are adopted from the open source program developed by Erkmen (2001). A set of equilibrium equations $[K] \{U\} = \{R\}$ is solved to obtain nodal displacements $\{U\}$, in which $\{R\}$ is the external load vector.

Internal forces are calculated based on nodal displacements. The algorithm of the program for static analysis is shown in Fig. 2.2.

2.2.2 Linear time-integration analysis

A set of linear dynamic equilibrium equations is solved in the program. This is illustrated below. First,

$$\{R_t\} = [M]\{\ddot{U}_t\} + [C]\{\dot{U}_t\} + [K]\{U_t\} \quad (2.2)$$

is solved by Newmark's direct integration method (Clough and Penzien 1993), where $\{R_t\}$, $\{\ddot{U}_t\}$, $\{\dot{U}_t\}$ and $\{U_t\}$ are the external load, acceleration, velocity and displacement vectors, respectively and $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices, respectively. The damping matrix $[C]$ is calculated based on the Rayleigh assumption in terms of $[M]$ and $[K]$ (Clough and Penzien 1993). The time-integration converts the dynamic equilibrium to linear algebraic equations by dividing total response time to finite time intervals.

$$\{\Delta R_t\} = [M]\{\Delta \ddot{U}_t\} + [C]\{\Delta \dot{U}_t\} + [K]\{\Delta U_t\} \quad (2.3)$$

$\{\Delta R_t\}$ is the incremental earthquake load which is calculated based on the ground acceleration record. $\{\Delta \ddot{U}_t\}$, $\{\Delta \dot{U}_t\}$ and $\{\Delta U_t\}$ are incremental acceleration, incremental velocity and incremental displacement vectors, respectively.

The following relationships can be established based on constant acceleration assumption between the vectors, $\{\Delta \ddot{U}_t\}$, $\{\Delta \dot{U}_t\}$ and $\{\Delta U_t\}$ (Clough and Penzien 1993)

$$\{\Delta \dot{U}_t\} = \frac{2}{\Delta t} \{\Delta U_t\} - 2\{\dot{U}_{t-\Delta t}\} \quad (2.4)$$

$$\{\Delta \ddot{U}_t\} = \frac{4}{\Delta t^2} \{\Delta U_t\} - \frac{4}{\Delta t} \{\dot{U}_{t-\Delta t}\} - 2\{\ddot{U}_{t-\Delta t}\} \quad (2.5)$$

In the above expressions, $\{\ddot{U}_{t-\Delta t}\}$ and $\{\dot{U}_{t-\Delta t}\}$ are the acceleration and velocity vectors for the previous time step. By substituting Eqs. 2.4 and 2.5 into Eq. 2.3 one obtains;

$$\{\Delta R_{t,eff}\} = [K_{eff}] \{\Delta U_t\} \quad (2.6)$$

In equation (2.6) the effective incremental load vector is;

$$\begin{aligned} \{\Delta R_{t,eff}\} &= \{\Delta R_t\} + [M] \left(\frac{4}{\Delta t} \{\dot{U}_{\Delta t-t}\} + 2\{\ddot{U}_{\Delta t-t}\} \right) + [C] (2\{\dot{U}_{\Delta t-t}\}) \\ [K_{eff}] &= [K] + \frac{2}{\Delta t} [C] + \frac{4}{\Delta t^2} [M] \end{aligned} \quad (2.7)$$

From the solution of Eq. 2.6, $\{\Delta U_t\}$ is determined. Displacement, velocity and acceleration vectors for the current time step are calculated for use in the next time step.

$$\{U_t\} = \{U_{t-\Delta t}\} + \{\Delta U_t\} \quad (2.8)$$

$$\{\dot{U}_t\} = \{\dot{U}_{t-\Delta t}\} + \{\Delta \dot{U}_t\} \quad (2.9)$$

$$\{\ddot{U}_t\} = \{\ddot{U}_{t-\Delta t}\} + \{\Delta \ddot{U}_t\} \quad (2.10)$$

The solution procedure for linear dynamic analysis is illustrated in Fig.2.3

2.2.3 Nonlinear Time-Integration Analysis

The differences between nonlinear and linear time-integration analysis can be summarized as follows;

- The stiffnesses of elements may not remain the same at the end of each time step because of material nonlinearity. Consequently, the stiffness matrix $[K]$ is recalculated when there is change in element stiffness. The calculation of element stiffness that includes material nonlinearity is explained in Section 2.2.3.1.
- When changes in stiffnesses occur within a time step, the internal forces calculated at the beginning of that step are corrected by taking into account the new element stiffness matrices. This procedure is explained in Section 2.2.3.2.

- Dynamic equilibrium is not satisfied when there are differences in internal forces calculated at the beginning and end of a time step. The correction of these unbalanced forces is explained in Section 2.2.3.3.

The program algorithm for nonlinear time-integration analysis is illustrated in Fig. 2.4. The initial stiffness dependent damping is used in the calculation of damping matrix [C], which is calculated only for the first time step and kept constant for the rest of the nonlinear analysis.

2.2.3.1 The element stiffness for flexure

All plastic deformations of beams due to flexure are assumed to be lumped in nonlinear rotational springs at the ends of a perfectly elastic beam, as shown in Fig.2.5. The stiffness coefficients k_{11} , k_{14} , k_{41} and k_{44} for the beam with rotational springs are the same as those for the elastic beam, since the rotational springs do not affect axial behavior. For bending behavior, the stiffness coefficients have to be modified. A sample calculation for stiffness coefficient k_{33} for a beam with end springs is illustrated in the Appendix 1.

2.2.3.2 Calculation of internal force increments

The calculation of internal force increments are explained below for a perfectly elastic beam with two flexural nonlinear springs at member ends. The explanatory example shows the case of stiffness change due to ‘yielding’, but depending on the model, every reloading and unloading may cause a change in stiffness. However, the calculation of the internal forces would be similar to the process explained below.

- **When there is no yielding during current time step**
 1. Inelastic stiffnesses of springs are assumed to be very large and they have no contribution to the element stiffness matrix, [k].

$$\{\Delta F_t\} = [k]\{\Delta U_t\} \quad (2.11)$$

$\{\Delta F_t\}$ and $\{\Delta U_t\}$ are incremental internal force and displacement vectors at time t.

2. Internal forces at time t are calculated by adding internal forces calculated in the previous time step $t-\Delta t$ and the internal force increments generated during Δt .

$$\{F_t\} = \{F_{t-\Delta t}\} + \{\Delta F_t\} \quad (2.12)$$

- **When there is yielding at both ends of the beam during current time step**

1. Internal force increments are calculated with the previous element stiffness matrix $[k_1]$ for the beam at the beginning of the time step. At this stage, there is no information whether there will be a change in the stiffnesses of springs.

$$\{\Delta F_{t,1}\} = [k_1] \{\Delta U_{t,1}\} \quad (2.13)$$

2. Internal forces at time t are calculated by adding internal forces calculated in the previous time step $t-\Delta t$, and the internal force increments at time, t .

$$\{F_{t,1}\} = \{F_{t-\Delta t}\} + \{\Delta F_{t,1}\} \quad (2.14)$$

In order to set the moment-rotation relationship for rotational springs at each node, internal forces M (bending moment) calculated in vector $\{F_{t,1}\}$ and spring rotations (θ) , calculated as the division of moment increments to spring stiffnesses, are used. The moment-rotation relationships for springs at nodes i and j are shown in Fig. 2.6.

3. If bending moments are greater than the corresponding yield values, the percentage of correctly calculated moment is established for each end ($p_{i,1}$ and $p_{j,1}$). This is calculated by dividing the moment increment up to yield ($M_y - M_{t-\Delta t}$) by total moment increment, ΔM_t , at each node.

$$p_{i,1} = \frac{M_{y,i} - M_{t-\Delta t,i}}{\Delta M_{t,1,i}} \quad (2.15)$$

$$p_{j,1} = \frac{M_{y,j} - M_{t-\Delta t,j}}{\Delta M_{t,1,j}} \quad (2.16)$$

4. The minimum percentage among $p_{i,1}$ and $p_{j,1}$ is selected as p_1 . Then the displacement vector is multiplied by this percentage to incorporate the contribution of the correct part of internal force increment vector calculated at the beginning of the step.

$$\{\Delta U'_{t,1}\} = p_1 \{\Delta U_{t,1}\} \quad (2.17)$$

$$\{\Delta F'_{t,1}\} = [k_1] \{\Delta U'_{t,1}\} \quad (2.18)$$

$$\{F'_{t,1}\} = \{F_{t,1}\} + \{\Delta F'_{t,1}\} \quad (2.19)$$

Then, the internal forces/bending moments and related spring rotations are calculated with new internal force increments, as shown in Fig. 2.7. The bending moment at one of the nodes is equal to the yield moment (this is illustrated at node i in Fig. 2.7.), depending on which end produced the minimum percentage. The other end is reduced to a value in the elastic zone.

5. At the node where yielding occurs (node i), the spring stiffness changes to its post-yield value.
6. The element stiffness matrix $[k_2]$ is recalculated with the post-yield stiffness value of spring at node i and the elastic stiffness value of spring at node j . Then, the rest of the internal force increments and the new internal forces are calculated. The related moment-rotation relationship is shown in Fig.2.8.

$$\{\Delta U_{t,2}\} = (1 - p_1) \{\Delta U_{t,1}\} \quad (2.20)$$

$$\{\Delta F_{t,2}\} = [k_2] \{\Delta U_{t,2}\} \quad (2.21)$$

$$\{F_{t,2}\} = \{F'_{t,1}\} + \{\Delta F_{t,2}\} \quad (2.22)$$

7. The bending moments at both ends are greater than their yield value. Again, the percentages of the correctly calculated moments at both ends are calculated as;

$$p_{i,2} = \frac{M_{y,i} - M_{t-\Delta t,i}}{\Delta M_{t,2,i}} \quad (2.23)$$

$$p_{j,2} = \frac{M_{y,j} - M_{t-\Delta t,j}}{\Delta M_{t,2,j}} \quad (2.24)$$

Since the yielding has already been taken into account at node i ; the percentage calculated at this node is equal to 1.0 and the minimum percentage p_2 is equal to the value calculated for node j .

8. The displacement vector is reduced this time by p_2 and the new internal force increment vector, the internal force vector and related spring rotations are calculated;

$$\{\Delta U'_{t,2}\} = p_2 \{\Delta U_{t,2}\} \quad (2.25)$$

$$\{\Delta F'_{t,2}\} = [k_2] \{\Delta U'_{t,2}\} \quad (2.26)$$

$$\{F'_{t,2}\} = \{F'_{t,1}\} + \{\Delta F'_{t,2}\} \quad (2.27)$$

The bending moment at node i , is reduced but it is still greater than its yield value, therefore there is no change in stiffness. On the other hand, the bending moment at node j is recently reached its yield value; therefore the element stiffness matrix is recalculated with post-yield values of spring stiffnesses at both ends. The moment-rotation relationship is shown in Fig. 2.9.

9. After the calculation of the new element stiffness matrix $[k_3]$, the internal force increments at both ends are calculated for the rest of the displacement, as shown in Fig.2.10.

$$\{\Delta U_{t,3}\} = (1 - p_2) \{\Delta U_{t,2}\} \quad (2.28)$$

$$\{\Delta F_{t,3}\} = [k_3] \{\Delta U_{t,3}\} \quad (2.29)$$

$$\{F_{t,3}\} = \{F'_{t,2}\} + \{\Delta F_{t,3}\} \quad (2.30)$$

10. Since yielding at both ends have been taken into account, the program starts the next step by using the element stiffness matrix calculated during the current step.

The process of the calculation of internal forces explained here was arranged for a beam with two nonlinear springs; therefore there were only two stiffness checks and changes (if needed) during the time step, one for each spring. It has to be noted that, for structures of multiple elements, the stiffness checks and changes have to be performed for all springs, during each time step.

2.2.3.3 Unbalanced load vector

The assumption of linear structural behavior during a given time step is not correct when a stiffness change occurs (e.g. yielding, unloading) within the time step. In this case, the dynamic equilibrium is not satisfied at the end of the step. When the stiffness of a node changes during a time step, the internal force (internal moment) calculated at this node at the beginning of the step, $\{F_{t,1}\}$, is not equal to the internal force at that same node at the end of that same step, $\{F_{t,3}\}$. The difference in these two internal forces gives the unbalanced internal force. The nodal unbalanced load vector is determined by calculating the unbalanced forces (moments) at the end of each element and then accumulating them at the same node. The unbalanced load vector is added to dynamic equilibrium during the next step to introduce the required correction for equilibrium.

$$\{\Delta R_{t,eff}\} = \{\Delta R_t\} + [M] \left(\frac{4}{\Delta t} \{\dot{U}_{\Delta t-t}\} + 2\{\ddot{U}_{\Delta t-t}\} \right) + [C] (2\{\dot{U}_{\Delta t-t}\}) - \{RU_t\} \quad (2.31)$$

The method is called 'Unbalanced Force Correction (UFC) Method. (Erkus 2004).

2.2.3.4 Hysteretic models used for members reinforced with steel and FRP

There are two hysteretic models adopted in the program; first one is for the steel reinforced elements developed by Clough (1966), and the second one is for the FRP reinforced elements developed by Sharbatdar and Saatcioglu (2003). The main characteristics of the Clough model are that the response aims toward the previous maximum response point and the unloading remains parallel to the initial elastic slope, as shown in Fig.2.11.

Sharbatdar and Saatcioglu (2003) developed an analytical model for hysteretic moment-displacement relationship of FRP reinforced concrete columns under constant axial compression. The primary moment-flexural displacement relationship defines the strength boundary and initial stiffness, as illustrated in Fig. 2.12(a). The primary curve has a tri-linear relationship. The first segment represents the initial elastic stiffness and aims at a point where the extreme compression fiber strain reaches $\varepsilon = \varepsilon_0/2$, where ε_0 is the strain at peak compressive stress f_c and can be taken as 0.002 for normal-strength concrete. The second segment represents the cracked stiffness up to the beginning of cover spalling at a maximum compressive fiber strain of ε_0 . The peak point on the primary curve corresponds to the failure of FRP reinforcement either due to buckling in compression or rupturing in tension.

The main difference of the hysteretic model for FRP reinforced elements compared to the Clough (1966) model for steel reinforced concrete elements is that the unloading branches of the FRP model tend to pass close to the origin. The rules defining unloading and reloading stiffnesses within the primary curve are defined as follows, relative to Fig. 2.12;

- Initial loading as well as subsequent loadings beyond the previous maximum displacement follows the primary curve.
- Positive and negative reloading and unloading within a displacement range of $\pm \Delta_1$ follows the initial stiffness.

- Unloading from a point between Δ_1 and Δ_3 aims at a point on the primary curve in the opposite direction which has a displacement equal to the average of the two previous displacements in the opposite direction, as shown in Fig.2.12(b). If the previous displacement in the opposite direction has not exceeded Δ_1 , unloading aims at the point with Δ_1 displacement on the opposite primary curve. If the previous displacement in the opposite direction has exceeded Δ_1 only once, unloading would aim at a point with displacement equal to the average of the maximum previous displacement and Δ_1 . Reloading follows the same slope as unloading until the primary curve is reached.
- Beyond Δ_3 , the member starts failing either by compression buckling or tension rupturing of FRP reinforcement. If the failure is due to the buckling of FRP bars, the element continues to resist loads due to the concrete strength until Δ_4 where the element fails due to the rupture of FRP bars. FRP reinforcement which has failed in compression can not maintain its tensile resistance.

2.2.3.5 Post-yield stiffness of flexural springs and chord rotations

The inelastic (post-yield) stiffness of flexural springs is determined by;

$$k_{in,fl} = p k_{el,fl} \quad (2.32)$$

Where, $k_{in,fl}$ and $k_{el,fl}$ are inelastic and elastic spring stiffnesses, respectively and p is the strain hardening ratio. The strain hardening ratio is determined by (Shooshtari 1998);

$$p = \frac{3EI}{L \left(1 - \frac{\xi}{2}\right) k_{el,fl}} \left(\frac{r}{1-r} \right) \quad (2.33)$$

where; r is the strain hardening ratio of the real beam and ξ is the ratio of the two end moments, defining the location of the point of inflection for the beam. The strain hardening ratio of the real beam and the location of the point of inflection are determined by the user. When the end moments are equal, the point of inflection is in the middle of

the beam and $\xi=1.0$. In the case of a cantilever beam, the point of inflection is at the fixed end and $\xi=0.0$. The chord rotations are defined in Fig. 2.13 for cantilever and frame elements. The total chord rotation of an element can be calculated as the sum of chord rotations of the elastic beam and the corresponding spring;

$$\theta_{ch} = \frac{ML}{6EI} + \theta_{sp} \text{ (for frame element)} \quad (2.34)$$

$$\theta_{ch} = \frac{ML}{3EI} + \theta_{sp} \text{ (for cantilever element)} \quad (2.35)$$

where; θ_{ch} is the chord rotation of an element and θ_{sp} is the rotation of corresponding spring.

2.2.3.6 Damping

Damping matrix is a combination of mass-dependent and stiffness-dependent effects as;

$$[C] = \alpha[M] + \beta[K] \quad (2.36)$$

Where, α and β are constants which can be calculated as;

- If only mass-dependent damping is assumed

$$\alpha = \frac{4\pi\xi_i}{T_i} \quad (2.37)$$

- If only stiffness-dependent damping is assumed

$$\beta = \frac{\xi_i T_i}{\pi} \quad (2.38)$$

- If both mass-dependent and stiffness-dependent damping are assumed

$$\alpha = \frac{4\pi(T_j \xi_j - T_i \xi_i)}{T_j^2 - T_i^2} \quad (2.39)$$

$$\beta = \frac{T_i T_j (T_j \xi_i - T_i \xi_j)}{\pi(T_j^2 - T_i^2)} \quad (2.40)$$

Here, ξ_i and ξ_j are the critical damping at i^{th} and j^{th} modes and T_i and T_j are the fundamental periods of the structure at i^{th} and j^{th} modes.

2.2.4 Three dimensional analysis

Three dimensional dynamic analysis can be conducted with Program SEQUAKE which considers the effects of the ground motions in all three global directions. Thus, the coupling in two orthogonal horizontal directions due to torsion is considered. However it should be noted that the plastic hinges at the ends of the members are formed separately based on the local principal axes of an element and the interaction between the end bending moments of an element is not considered in the yield criterion. This simplifying assumption may be acceptable for beams, which essentially bend in one direction independent of bending of the beams in the orthogonal direction. However, the hysteretic behaviour of vertical elements (columns) does not include the effects of biaxial bending, and hence only provides approximate results.

2.3. Verification of SEQUAKE

Two examples were used to verify the computer program SEQUAKE against program DRAIN-RC, a program originally developed by Kanaan and Powell (1973) as DRAIN-2D for dynamic analysis of plane structures. Alsawiat (1993) modified the original version of DRAIN-2D by adding new hysteretic models for shear and anchorage slip. This version was then further modified by Shooshtari (1998) who added a hysteretic model for infill wall elements and created a version applicable only to reinforced concrete structures, called DRAIN-RC. The hysteretic models incorporated include; Takeda (1970) for flexure (modified to have a bilinear primary curve), Ozcebe and Saatcioglu (1989) for shear and Alsaiwat and Saatcioglu (1992) for anchorage slip. The main differences between the models proposed by Clough (1966), which was adopted for SEQUAKE and Takeda (1970), which was adopted in DRAIN-RC are that the later includes stiffness changes at flexural cracking also and describes a tri-linear primary curve and that in the model of Takeda the unloading stiffness is reduced by an exponential function of the previous maximum deformation. Later, a simplified Takeda

hysteresis model was proposed by Powell (1975), using a bilinear primary curve. In Drain-2D, two versions of simplified Takeda models are present. The one which includes the reduction in the unloading curve and the more simplified one which does not include this reduction, thereby becoming the same as that proposed by Clough. In the analyses, the modified version of Takeda's model (essentially following the Clough model) was used. Since SEQUAKE only considers inelasticity in flexure for steel reinforced concrete elements, inelastic deformations for shear and anchorage slip are suppressed in DRAIN-RC for proper comparison. The exact solution of a single-degree-of freedom (SDOF) structure is also used to verify the linear time-history analysis of SEQUAKE. Furthermore, two additional examples were used to verify three-dimensional nonlinear analyses of SEQUAKE.

2.3.1 Two dimensional linear time-history analysis

The dynamic equilibrium equation of a SDOF system subjected to a load formed from linear segments is as follows;

$$KX + C\dot{X} + M\ddot{X} = At + B \quad (2.41)$$

Where, K , C and M are the stiffness, damping and mass of the system. Displacement, velocity and acceleration at each time, t , are represented by X , $\dot{X}$ and $\ddot{X}$. Each linear segment of the loading is defined by A and B . Complementary solution, X_c , of this equation is given by;

$$X_c = e^{-\xi\omega t} [d_1 \cos(\omega_d t) + d_2 \sin(\omega_d t)] \quad (2.42)$$

where, ξ is the ratio of the given damping, c , to the critical value, c_c ;

$$\xi \equiv \frac{c}{c_c} = \frac{c}{2m\omega} \quad (2.43)$$

Undamped and damped circular velocities are expressed with ω and ω_d . Particular solution of the equation is;

$$X_p = \frac{A}{K}t + \frac{B}{K} - \frac{AC}{K^2} \quad (2.44)$$

The expressions d_1 and d_2 in Eq. (2.42) become;

$$d_1 = D - \frac{B}{K} + \frac{AC}{K^2} \quad (2.45)$$

$$d_2 = \frac{V - \frac{A}{K} + \xi\omega \left(D - \frac{B}{K} + \frac{AC}{K^2} \right)}{\omega_d} \quad (2.46)$$

Where, D and V are the initial displacement and velocity when $t=0$. Exact solution is the sum of complementary and particular solutions.

$$X = X_c + X_p \quad (2.47)$$

A water tower subjected to a seismic load is analyzed using exact solution, SEQUAKE and DRAIN-RC. Mass and stiffness dependent damping ($C=\alpha M+\beta K$) is used for the example. The structure and the loading are shown in Fig. 2.14 and 2.15. The displacement responses are compared in Fig. 2.16.

2.3.2 Two dimensional nonlinear time-history analysis

The water tower and a five storey plane frames are used to verify the nonlinear analysis of SEQUAKE against DRAIN-RC. Mass and stiffness dependent damping ($C=\alpha M+\beta K$) is used for both examples. The artificial ground acceleration records generated by Atkinson and Beresnev (1998) are used as ground motions for both examples. Ottawa record reflects short-period hazard with an event of M6.0 and a maximum acceleration of

0.334g. Vancouver record reflects short-period hazard with an event of M6.5 and a maximum acceleration of 0.537g. The artificial records are plotted in Fig. 2.17.

Water Tower

The nonlinear behavior of the water tower used in the previous example is analyzed under the Ottawa record, using both SEQUAKE and DRAIN-RC. The results of analyses for top lateral displacement and top total rotation are shown in Figs. 2.18 and 2.19. Fig. 2.20 shows the flexural moment-chord rotation hysteretic relationship of the water tower.

Five storey frame

A five storey plane frame is analyzed with DRAIN-RC and SEQUAKE under the Vancouver record. The frame and element geometry are shown in Fig. 2.21. Most of the beam and column springs experienced nonlinear deformations which are represented with gray dots in Fig. 2.22. The lateral top displacement responses are compared with DRAIN-RC in Fig. 2.23. Fig. 2.24 shows the moment-chord rotation relationship of one of the beams in the first storey.

2.3.3 Three dimensional nonlinear time-history analysis

A three-dimensional model of a two storey symmetric concrete structure, reinforced with steel rebars, shown in Fig. 2.25 was selected to verify SEQUAKE. The structure was subjected to the same earthquake in both horizontal X and Z directions and the mass is assumed to be lumped at the center of rigidity of each floor. The displacement response in one direction was compared with that in the other direction. It was found that the responses in both directions of this symmetric structure were identical, as shown in Fig. 2.26. In addition, the same structure was modeled as series of two-dimensional frames linked together (Fig. 2.27) as in the 5 storey building example used in Section 2.3.2, since the two-dimensional nonlinear analysis was already verified in that section. It was observed that the top lateral displacement response of two-dimensional model was the same as that of three-dimensional model as illustrated in Fig. 2.28. Because the three dimensional analysis was conducted with the same ground motion applied in two orthogonal directions and the hysteretic model for columns is identical in both directions,

this comparison provided an indication of proper functioning of the program under three dimensional ground excitations.

The above analysis of the same structure was conducted once again under eccentric loading conditions. The centre of mass at each floor level was moved 0.6m from the center of rigidity of each floor in both X and Z directions to create torsional effects, as shown in Fig.2.29. The ground motion was applied separately in each direction. It was found that top lateral displacement response in one direction was the same as the response in the orthogonal direction (Fig. 2.30). However because of the torsion effect, the analysis results in this case were different than those of the structure with no eccentricity. It was concluded that SEQUAKE can be used for nonlinear analyses of three-dimensional structures reinforced with steel and FRP re-bars except that the effects of biaxial bending were ignored in nonlinear analyses due to the fact that the hysteretic models developed for both steel and FRP reinforced members account for moments in each direction separately.

2.4. Anchorage slip effect

Deformations at member ends, under bending, consist of two components; i) flexural deformations along the length of the member and ii) rigid body rotation of the member at its end due to the anchorage slip of flexural reinforcement within the adjoining member. The latter component occurs when the critical section for flexure is located at the end of a member (Alsiwat and Saatcioglu, 1992). The penetration of reinforcement strain into the adjacent member, especially after the yielding of steel reinforcement and subsequent strain hardening, or high levels of straining of low modulus FRP bars results in the elongation and extension of reinforcement contributing to the interface crack. This results in rigid-body deformations that are not accounted for in flexural analysis. If the embedment length of reinforcement is not sufficiently long, and the straining of reinforcement extends to the embedded end of the reinforcement, local slippage of the bar may occur, even if the bar continues maintaining its full anchorage capacity. This results in increased bond stresses and additional movement of the bar within the adjoining

member. Anchorage slip provides softening due to the combined effects of reinforcement extension and slip within the adjoining member.

Slippage of reinforcement in concrete occurs when the bar is stressed up to the cutoff point and this takes place when the embedment length of the bar is not sufficiently long. Bar slip can be obtained from local bond-slip relationships. Some of the bond-slip relationships for FRP bars in literature were proposed by Malvar (1995), Cosenza et al. (1997), Tighiouart et. al. (1998), and Zhang and Benmokrane (2002). However, bar slip is generally either zero or small as compared to bar extension (Alsiwat and Saatcioglu, 1992) and this study focuses only on the elastic elongation of FRP bars.

2.4.1 Previous experimental research

Sharbatdar (2003) tested several cantilever columns at the University of Ottawa under reversed cyclic loading. The columns had a 355 mm square-cross section with either a 1000 mm or a 1900 mm length. This translated into shear spans of 1280 mm or 2180 mm, when the top-loading beam is considered and the column height was measured to the point of application of lateral forces. The longitudinal reinforcement was 9.5mm diameter Pultrall CFRP bars. Two reinforcement arrangements were used, consisting of either eight-bars or twelve-bars. The average tensile strength and modulus of elasticity obtained from coupon tests were 1470 MPa and 125000 MPa. The length of FRP bars inside the footing was approximately 470mm and the clear cover in all cases was 20mm. NEFMAC grids were used as transverse reinforcement with an 88 mm or 175 mm spacing. The concrete strength on the day of testing was determined to be 38 MPa.

Sharbatdar also tested full-scale cantilever beams under reversed cyclic and monotonic loading. The beams were 305 mm wide and 405 mm deep with two different lengths; 1000 mm and 1900 or 1980 mm. The shear spans were 870 mm and 1780 or 1870 mm respectively, measured to the point of application of lateral force. Six 9.5 mm diameter Pultrall CFRP bars were used as top reinforcement and four same size and type bars were used as bottom reinforcement. The length of FRP bars inside the footing was approximately 470mm and the clear cover in all cases was 15 mm. Grids were used as

transverse reinforcement with 180 mm or 90 mm spacing. The actual concrete strength on the day of testing was determined to be 40 MPa.

Experimental data of Sharbatdar (2003) are used to assess the significance of recorded levels of anchorage slip, and compared with the results obtained from the analytical work conducted as a part of the current study. Experimental data included total deformations at the ends of members, provided in the form of lateral force-lateral displacement relationships, and anchorage slip deformations, provided in the form of moment-anchorage slip rotations.

2.4.2 Analytical study

The analytical study includes the investigation of FRP reinforced concrete test specimens reported by Sharbatdar (2003) and the computation of deformations caused by anchorage slip for comparison with those obtained experimentally. Therefore, first moment-curvature relationships obtained from the experimental study were compared to those obtained analytically. Second, the moment-slip displacement relationships were calculated from the moment-slip rotations obtained experimentally and then compared with those obtained from the analytical study. Third, the lateral load-lateral displacement relationships provided by Sharbatdar were transformed to moment-lateral displacement relationships by multiplying lateral loads by shear spans and compared with the moment-total displacement relationships obtained from the analytical study.

For analytical predictions of anchorage slip deformations, first sectional analyses of sections used by Sharbatdar (2003) are selected, as summarized in Table 2.1. Moment-curvature relationships are established by using computer program RC-SECTION (1997). These relationships are transformed to tri-linear primary curves; where the first segment starts from the origin and continues to the point at which the concrete compressive strain is $\varepsilon_c=0.001$. The second segment starts from the end of the first segment and continues up to a point where the concrete compressive strain is $\varepsilon_c=0.002$. The third segment continues up to a point where either the strain in FRP rebar in tension reaches its rupturing value or the strain in FRP rebar in compression reaches its buckling value. Figs. 2.31 to 2.40 show

the comparison of experimentally recorded and analytically generated moment-curvature relationships for selected FRP reinforced concrete elements tested by Sharbatdar. Subsequently, flexural moment-displacement relationships are computed by integrating curvatures twice, between the member ends, for the analytical study. Anchorage slippage (extension of reinforcement within the adjoining member) is computed by integrating FRP strains along the development length of reinforcement. Total member tip displacements, consisting of flexural and anchorage slip components, and the resulting moment-total displacement relationships are obtained by summing flexural moment-displacement relationships and moment-anchorage slip displacement relationships. Moment-total displacement and moment-anchorage slip displacement relationships are also compared with those of obtained from the experimental study of Sharbatdar (2003). Further details are given in the following sections.

2.4.2.1 Displacement due to flexure

Rotations and deflections of a member are calculated by integrating curvatures between the ends of a member, at strain levels described above;

$$\theta_f = \int \phi dx \quad (2.48)$$

$$\Delta_f = \int x \phi dx \quad (2.49)$$

Where; θ_f and Δ_f are the rotation and displacement due to flexure, x is the longitudinal coordinate of a member (Park and Paulay, 1975).

Moment-flexural displacement relationships are calculated by integrating curvatures obtained from the analytical study. Consequently, these displacements are combined with the analytically obtained slip deformations in order to obtain the total displacements.

2.4.2.2 Extension in FRP reinforcement and displacements due to anchorage slip

Steel reinforcement may develop elastic and plastic regions within the embedment length upon penetration of yielding into the adjoining element. However, FRP reinforcement does not yield. Therefore, it only develops elastic strains under monotonic and cyclic loadings. The FRP reinforcing bar embedded in concrete and its strain distribution are illustrated in Fig. 2.41 (a) and (b).

The reinforcing bar should be embedded in the concrete at least for a length l_d in order to transmit its force to the concrete by bond (Park and Paulay, 1975). If the average bond stress uF for the FRP bar, which is the shear force per unit area of bar surface, is assumed constant, the following equilibrium can be written as illustrated in Fig. 2.42.

$$T = A_{frp} f_{frp} = uF \pi d_b l_d \quad (2.50)$$

where, T is tensile force in the bar, A_{frp} is the sectional area of FRP bar, f_{frp} is the tensile strength of FRP bar and d_b is the bar diameter. Sharbatdar (2003) suggested that an average bond stress equal to 5.5 MPa can be used for the Pultrall bars and concrete strength used in his experimental program. Consequently, the development length of a FRP bar for a given strain can be calculated as;

$$l_d = \frac{A_{frp} f_{frp}}{uF \pi d_b} = \frac{A_{frp} (E_{frp} \epsilon_{frp})}{uF \pi d_b} \quad (2.51)$$

The extension of FRP bar δ_{ext} , can be computed by integrating the strains over the development length l_d . Referring Fig. 2.41, this can be expressed as the area under strain diagram.

$$\delta_{ext} = \frac{\epsilon L_d}{2} \quad (2.52)$$

The corresponding rotation θ_{as} can be approximated by dividing the bar extensions δ_{ext} by the distance between the bar and the neutral axis as explained in the study of Saatcioglu, Alsiwat and Ozcebe (1992);

$$\theta_{as} = \frac{\delta_{ext}}{d - c} \quad (2.53)$$

The member end displacement Δ_{as} due to anchorage slip is the product of member end rotation θ_{as} and member length L , since the effect of anchorage slip is to induce rigid body rotation as follows;

$$\Delta_{as} = \theta_{as} L \quad (2.54)$$

The moment-slip displacement relationships comparing the analytical results of current study and those obtained based on experimental data by Sharbatdar (2003) are plotted in Fig. 2.43 to 2.52. Total tip displacement is calculated as the sum of displacement components due to flexure and anchorage slip.

$$\Delta_t = \Delta_f + \Delta_{as} \quad (2.55)$$

Moment-total displacement relationships are computed and plotted by employing the procedure described earlier. Figs 2.53 to 2.62 show the moment-total displacement relationships obtained from the analytical study to those obtained from the experimental form the study of Sharbatdar (2003).

2.4.3 Softening effect of anchorage slip on flexural response of FRP reinforced concrete members

The effect of anchorage slip on flexural response of reinforced concrete structures is to soften the structure. Additional deformations caused by rigid-body member-end rotations induced by anchorage slip result in reduced stiffness. In the current research project, this effect is accounted for through percentage reduction in flexural stiffness. The moment-total displacement relationships with and without the anchorage slip effect for the

sections reinforced with FRP considered in this analytical study are plotted in Figs. 2.63 to 2.72 in order to show the reduced stiffnesses and Table 2.2 summarizes the effects of anchorage slip on stiffness as the ratio of stiffnesses calculated taking into account the anchorage slip effect to the stiffnesses calculated by excluding the effect, for each region of the tri-linear primary force-deformation relationship. Consequently, a parametric investigation is carried out to establish percentage reduction in flexural stiffness caused by anchorage slip. Accordingly, a 300 mm by 300 mm square column section is selected with twelve 10 mm-diameter FRP bars. The section is assigned to four members with member lengths of 1m, 2m, 3m, and 4m. Each member is subjected to 0% P_o , 5% P_o , 10% P_o , 20% P_o , 30% P_o , 40% P_o and 50% P_o as axial compression, where P_o represents the member concentric capacity in compression. The stiffness reduction factors due to the softening caused by anchorage slip are plotted in Figs. 2.73 to 2.79 for each of the three segments of tri-linear force-displacement relationship (shown as MFS1, MFS2 and MFS3 for the first, second and third segments, respectively). It can be observed in these figures that the difference in the first slope with and without anchorage slip is very small. The influence of the anchorage slip increases in the second and third slopes. Also, the effect of anchorage slip is more significant when the applied axial loading or the member length decreases.

2.4.4 Anchorage slip in steel reinforced members

An analytical study similar to the one conducted for FRP reinforced members is also performed for steel reinforced members, in order to conduct a comparative study. The main difference between the steel and FRP reinforced concrete elements is that the steel reinforcement enters into the strain hardening range under increased loading, while the FRP reinforcement remains elastic. The steel reinforcing bar embedded in concrete and its strain distribution are illustrated in Fig. 2.80 (a) and (b) (Alsiwat and Saatcioglu, 1992). In Fig. 2.80 (b) the strain distribution is illustrated over a length which consists of elastic region L_e , yield plateau L_{yp} , strain hardening region L_{sh} and pull-out cone L_{pc} . The equilibrium in steel bars is given as follows:

$$T = A_s f_s = u_s \pi d_b l_d \quad (2.56)$$

Where, T is the tensile force in re-bar, A_s is the sectional area of steel bar, f_s is the tensile strength of steel bar and d_b is the bar diameter. The extension δ_{ext} is computed by integrating the strains developed in the elastic, yield plateau and strain hardening regions, assuming that pull-out is prevented. The length of elastic region, L_e , is equal to the development length l_d and can be calculated based on the expressions given in CSA Standard A23.3-2004 (2005). In this study, it is assumed that there is zero stress increment within the yield plateau, which results in the length of this zone to be zero. The length of strain hardening zone is calculated as follows (Alsiwat and Saatcioglu, 1992):

$$L_{sh} = \frac{\Delta f_s d_b}{4u_f} \quad (2.57)$$

Where; Δf_s is the difference between steel stresses at ultimate and yield, and the bond stress u_f is calculated from the expression suggested by Alsiwat and Saatcioglu (1992):

$$u_f = \left(5.5 - 0.07 \frac{S_L}{H_L} \right) \sqrt{\frac{f'_c}{27.6}} \quad (2.58)$$

Where; S_L and H_L are clear spacing and height of lugs on the bar, respectively.

Displacements due to flexure and anchorage slip in a steel reinforced concrete element are also calculated following Eq. 2.49 and 2.54 used earlier for FRP reinforced concrete elements. Total displacement of member is computed from Eq. 2.55.

A parametric study, similar to the one conducted earlier for FRP reinforced concrete columns, is conducted for steel reinforced concrete columns having a 300mm by 300mm square section with eight 20 mm-diameter steel bars. Four different member lengths and seven different levels of axial compression are used to assess the stiffness reduction caused by anchorage slip. The reduction factors obtained for the steel reinforced concrete columns are summarized in Figs. 2.81 to 2.87. A bilinear relationship is used to characterize the primary force-displacement relationship for steel reinforced concrete

members. In this case, the effect of anchorage slip is significant even for the first slope, contrary to that for FRP reinforced concrete elements.

2.4.5 Recommendations for stiffness softening due to anchorage slip

In order to take into account the effect of anchorage slip as stiffness softening, the reduction factors provided in Figs. 2.73 to 2.79 are recommended for FRP reinforced concrete elements. For instance, the first, second and third segment stiffnesses of a tri-linear force-displacement relationship of a six meters long FRP reinforced beam, can be reduced using the reduction factors given for the three meters long cantilever element with zero axial loading (Fig. 2.73). Accordingly, these factors can be taken as 0.9, 0.8 and 0.6 respectively.

Similarly, for a three meters long FRP reinforced column subjected to an axial loading which is equal to the 30 percent of its concentric capacity, Fig. 2.77 can be used. Accordingly, the reduction factors can be taken as 0.9, 0.7 and 0.5 for the first, second and third segment stiffnesses of its tri-linear force-displacement relationship.

In order to take into account the effect of anchorage slip as stiffness softening in steel reinforced concrete members, the reduction factors provided in Figs 2.81 to 2.87 are recommended. Accordingly, the factors for the first and second segment stiffnesses of the bi-linear force-displacement relationship of a six meters long steel reinforced concrete beam can be taken as 0.8 and 0.6 respectively. Same factors can be taken as 0.6 and 0.5 for a three meters long column, assuming that it is subjected to an axial loading which is equal to the 30 percent of its concentric capacity.

Table 2.1 Properties of the members used in both analytical and experimental studies

Specimen Label	f _{co} (Mpa)	Reinforcement	s(mm)	H(mm)	L (mm) (P-applied)	P(kN)	P/Po (%)	b (mm)	h (mm)	concrete cover(mm)
Column CFCL3	38	12-9.5 mm bar	175	1900	2180	1115	27	355	355	20
Column CFCL4	38	12-9.5 mm bar	88	1900	2180	1115	27	355	355	20
Column CFCL6	38	8-9.5 mm bar	88	1000	1280	1358	33	355	355	20
Column CFCL7	38	8-9.5 mm bar	88	1000	1280	679	17	355	355	20
Column CFCL8	38	12-9.5 mm bar	175	1000	1280	1220	30	355	355	20
Column CFCL9	38	12-9.5 mm bar	88	1000	1280	1220	30	355	355	20
Column CFCL10	38	12-9.5 mm bar	88	1000	1280	627	15	355	355	20
Beam CFB3	35	10-9.5 mm bar	88	1000	870			305	405	15
Beam CFB4	35	10-9.5 mm bar	185	1900	1780			305	405	15
Beam CFB6	35	10-9.5 mm bar	88	1900	1870			305	405	15

Table 2.2 Effect of anchorage slip on columns and beams

Specimen Label	First Slope	Second Slope	Third Slope	Reinforcement Ratio (%)	L (mm)	P/Po (%)
	k(f+s)/kf	k(f+s)/kf	k(f+s)/kf			
Column CFCL3	0.94	0.66	0.58	0.7	2180	27
Column CFCL4	0.94	0.66	0.57	0.7	2180	27
Column CFCL6	0.91	0.54	0.47	0.46	1280	33
Column CFCL7	0.89	0.58	0.59	0.46	1280	17
Column CFCL8	0.92	0.55	0.42	0.7	1280	30
Column CFCL9	0.91	0.55	0.41	0.7	1280	30
Column CFCL10	0.88	0.58	0.47	0.7	1280	15
Beam CFB3	0.76	0.54	0.34	0.39/0.26	870	
Beam CFB4	0.87	0.68	0.53	0.39/0.26	1780	
Beam CFB6	0.87	0.69	0.54	0.39/0.26	1870	

k(f+s):k(flexure+anchorage slip) kf:kflexure

strong side/weak
side

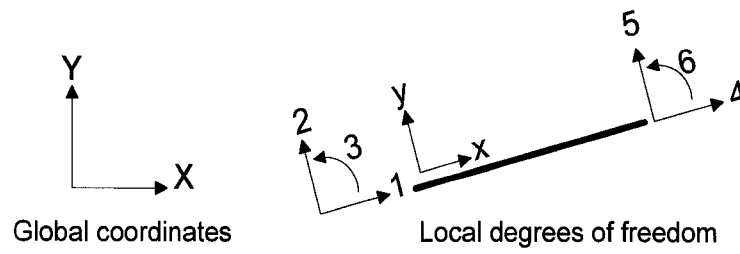


Figure 2.1 Local degrees of freedom

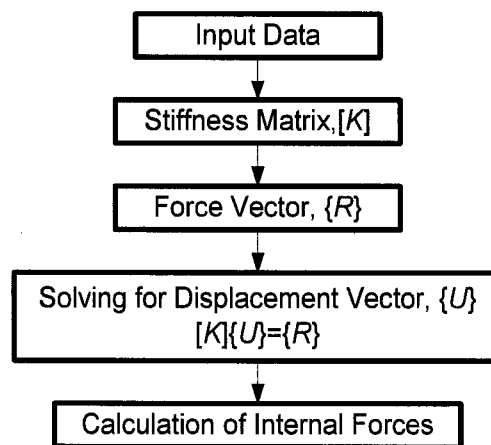


Figure 2.2 Algorithm of the program for static analysis

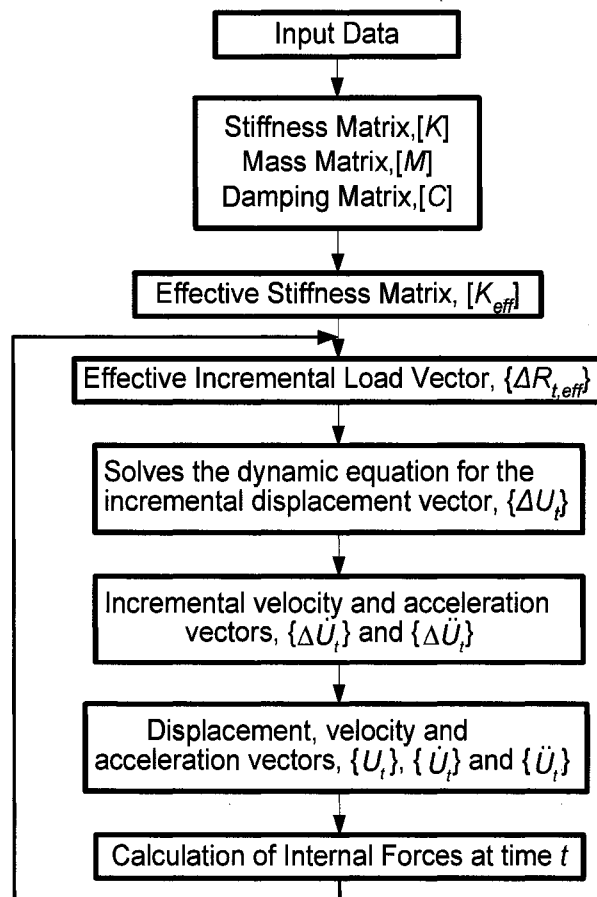


Figure 2.3 Algorithm of the program for linear time-integration analysis

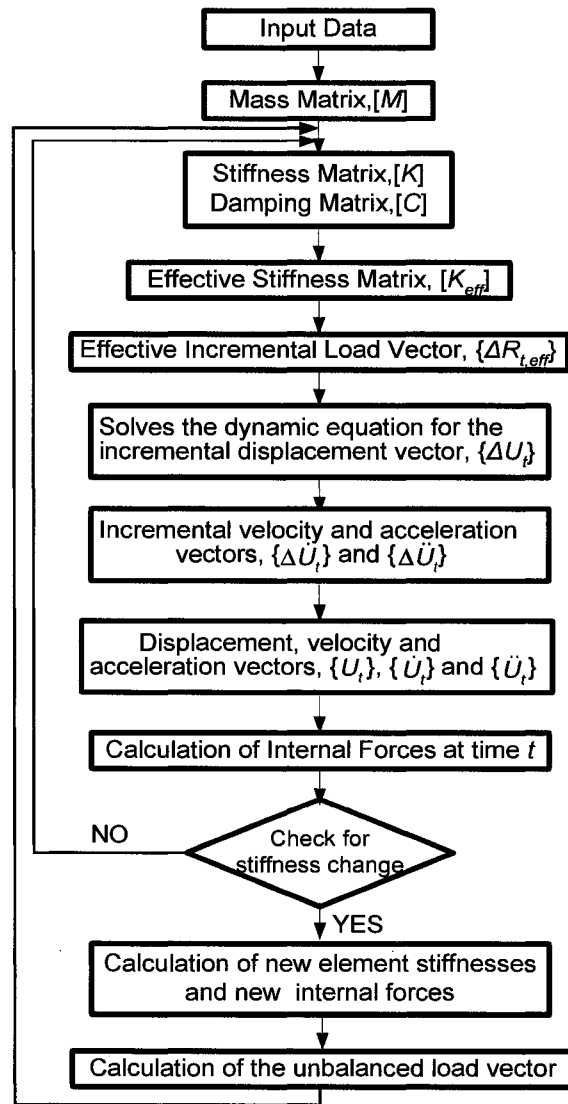


Figure 2.4 Algorithm of the program for nonlinear time-integration analysis

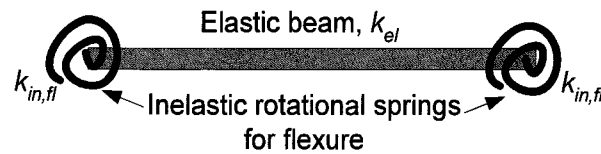


Figure 2.5 Elastic beam with nonlinear rotational springs for flexure

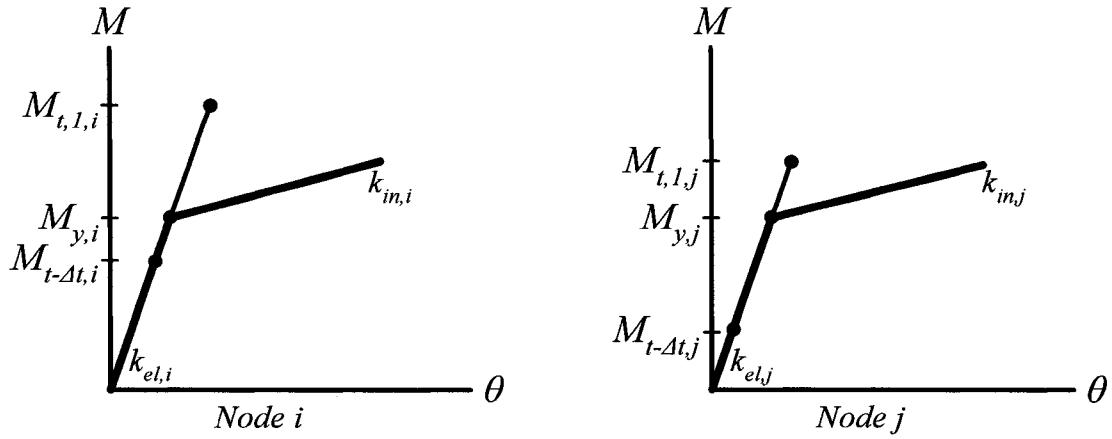


Figure 2.6 Moment-rotation relationships of springs showing moments calculated using their elastic stiffnesses at both ends i and j .

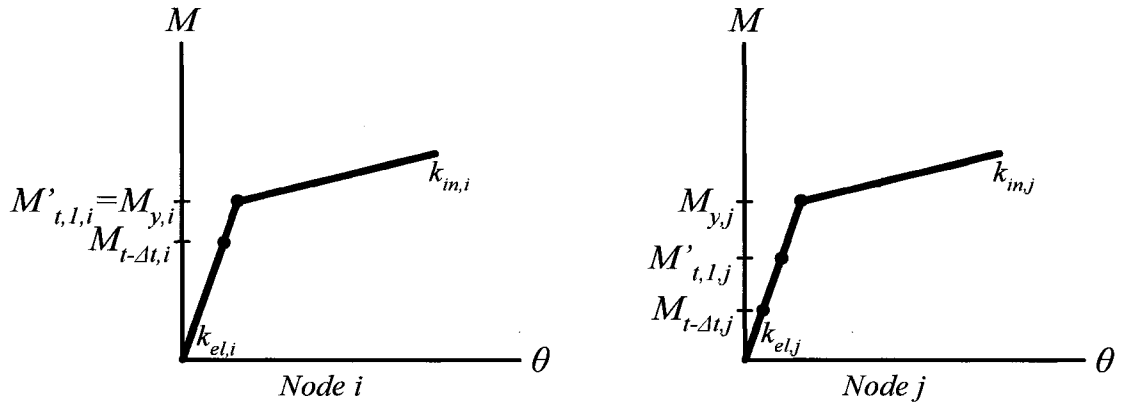


Figure 2.7 Moment-rotation relationships of springs showing moments reduced by p_1

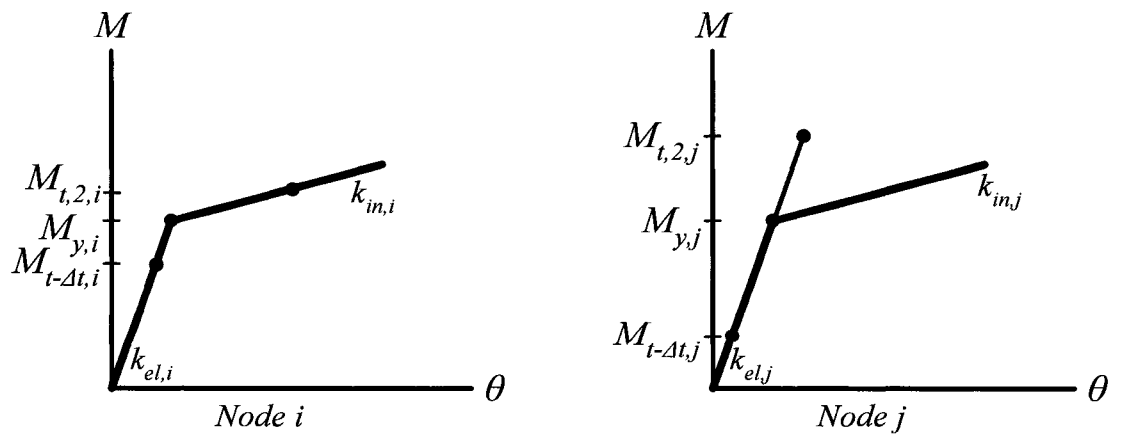


Figure 2.8 Moment-rotation relationships of springs showing moments calculated using its inelastic stiffness at node i and its elastic stiffness at node j .

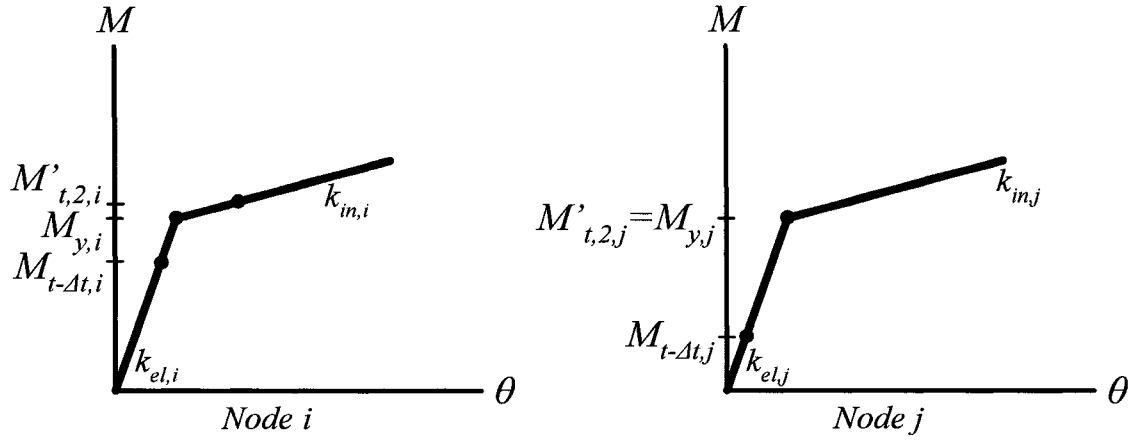


Figure 2.9 Moment-rotation relationships of springs showing moments reduced by p_2

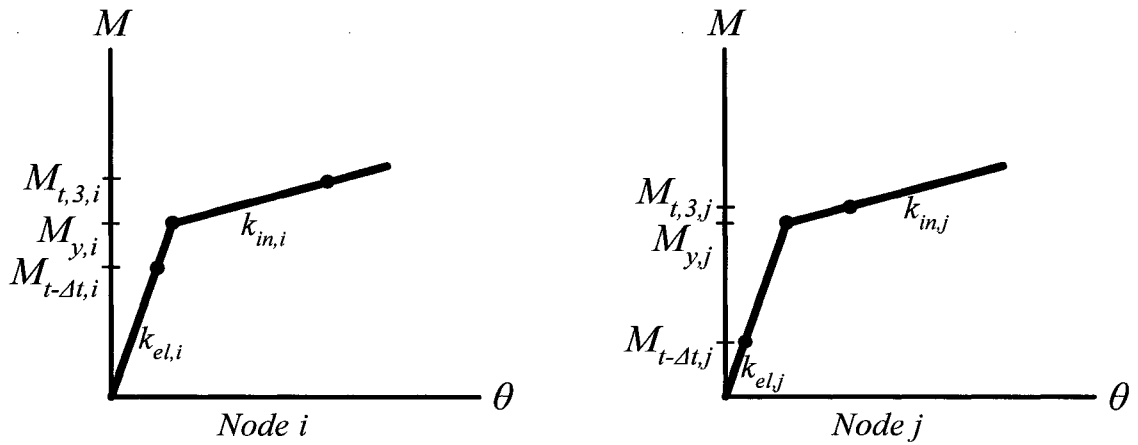


Figure 2.10 Moment-rotation relationships of springs showing moments calculated using their post-yield stiffnesses at both nodes i and j .

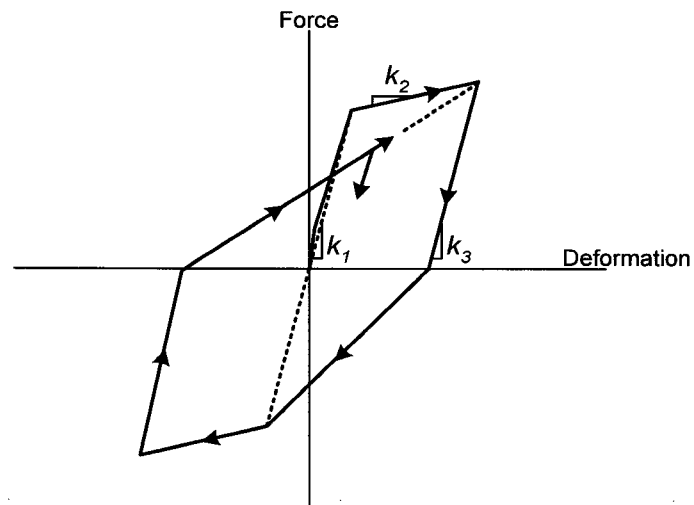


Figure 2.11 Stiffness degrading model by Clough (1966)

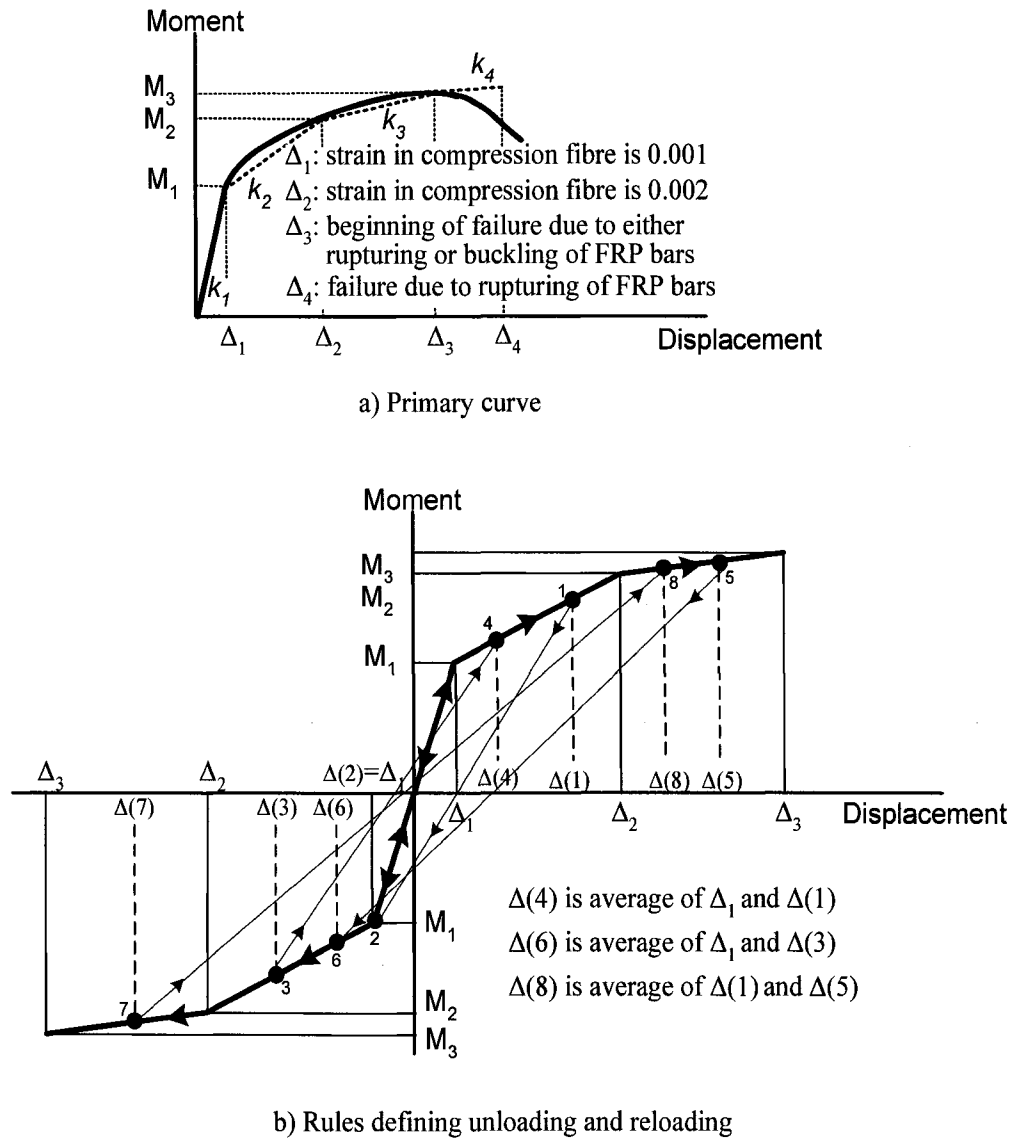


Figure 2.12 Moment-displacement model of Sharbatdar and Saatcioglu (2003)

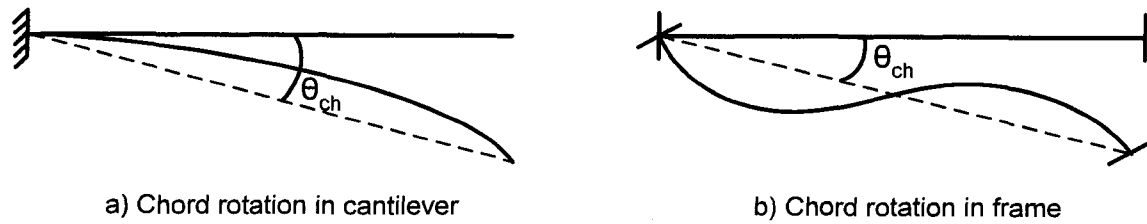


Fig. 2.13 Definition of chord rotation in cantilever and frame

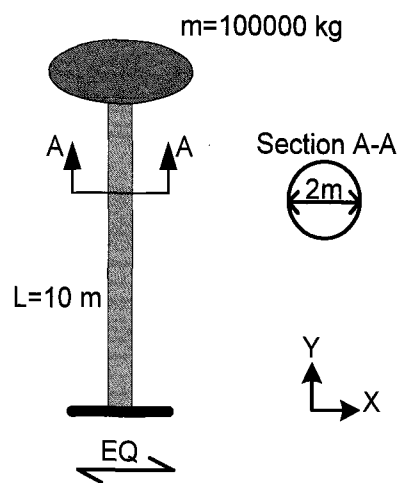


Figure 2.14 Water tower subjected to seismic load.

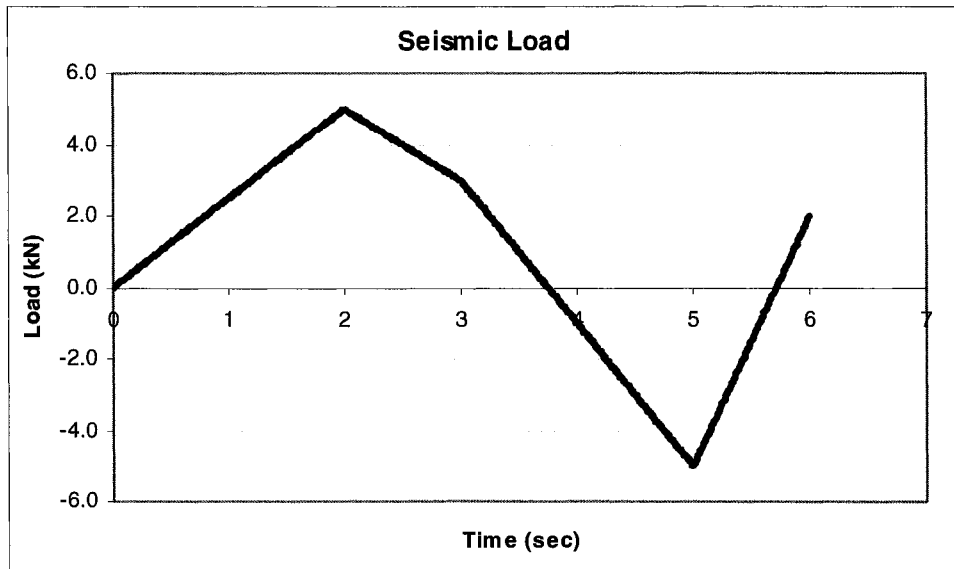


Figure 2.15 Seismic load applied to the water tower.

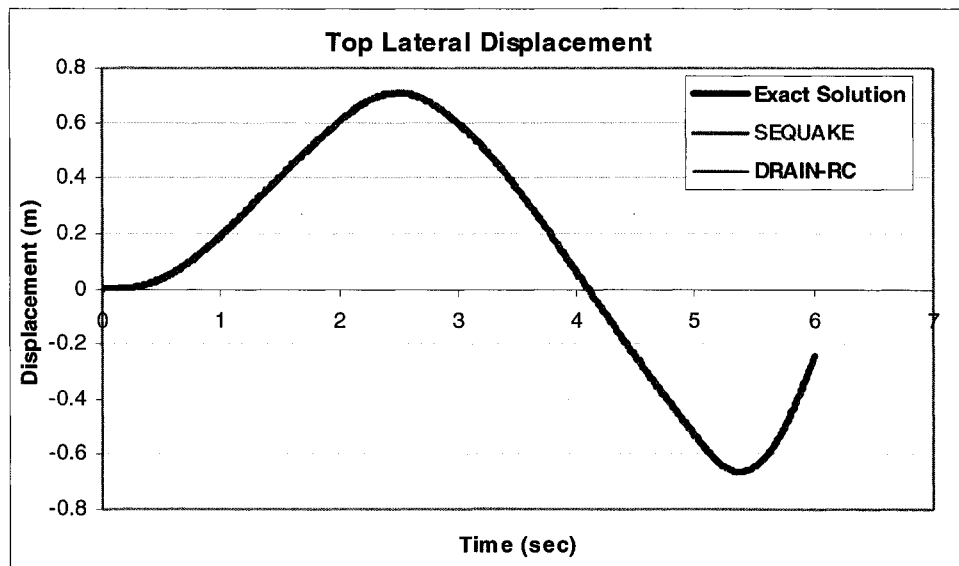


Figure 2.16 Displacement responses of the water tower.

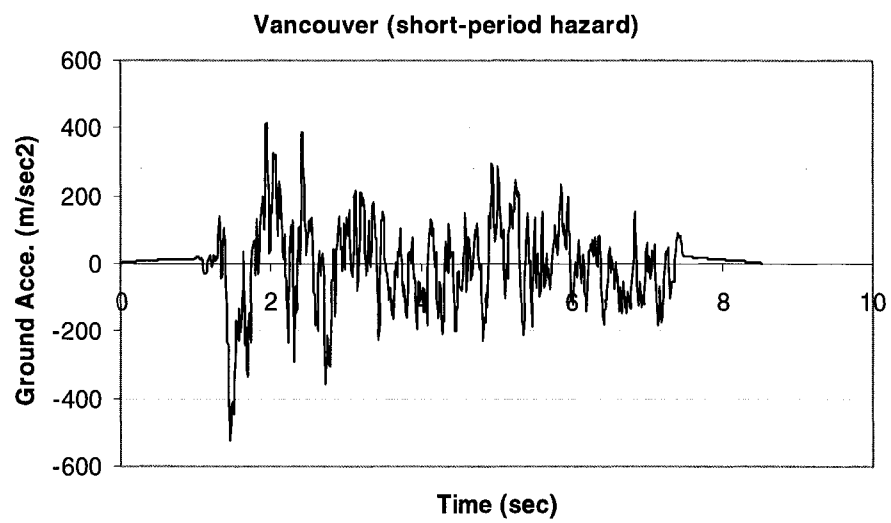
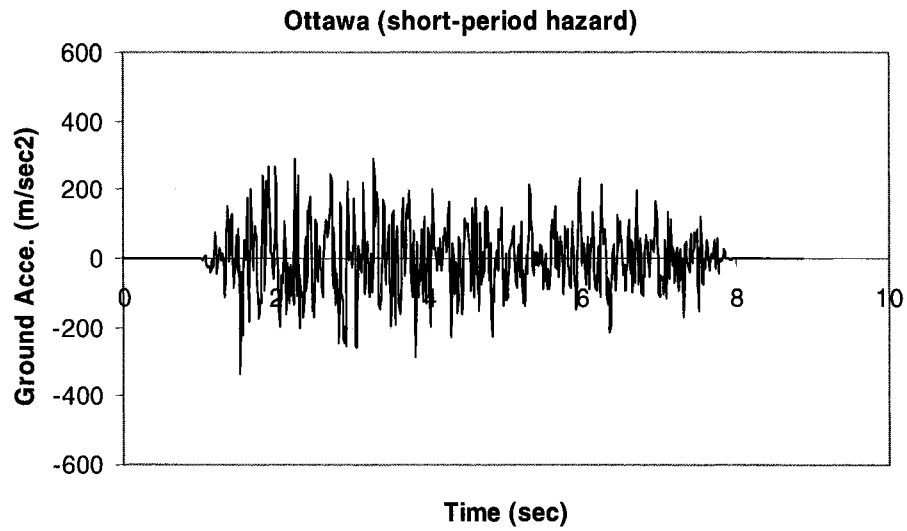


Figure 2.17 Artificial ground acceleration time histories used for examples

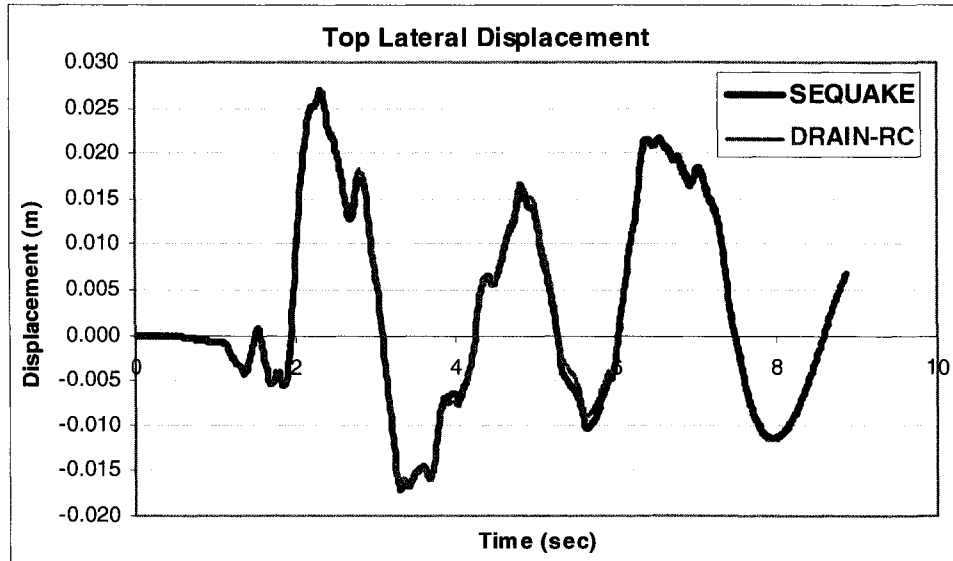


Figure 2.18 Top lateral displacement response of the water tower

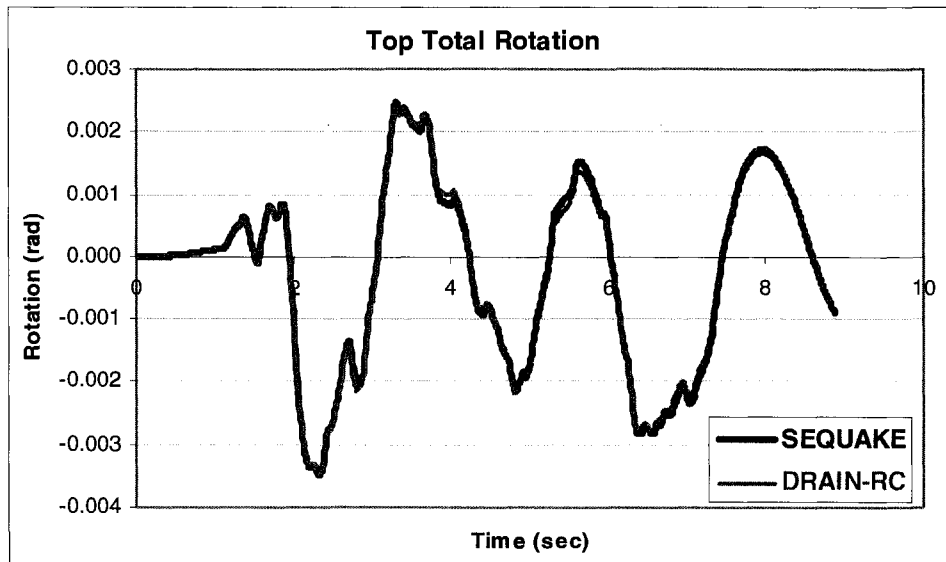


Figure 2.19 Top total rotation response of the water tower

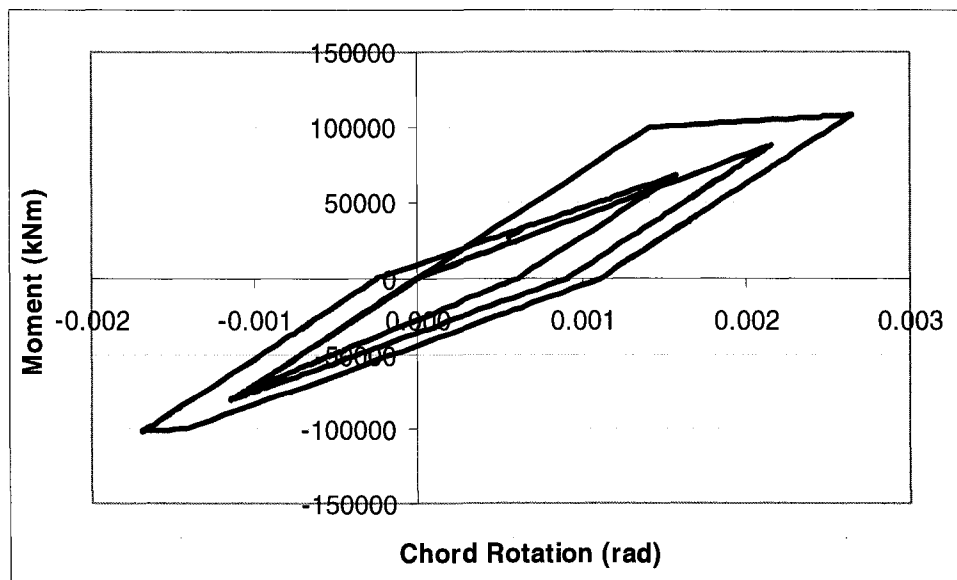


Figure 2.20 Flexural moment-chord rotation relationship of the water tower

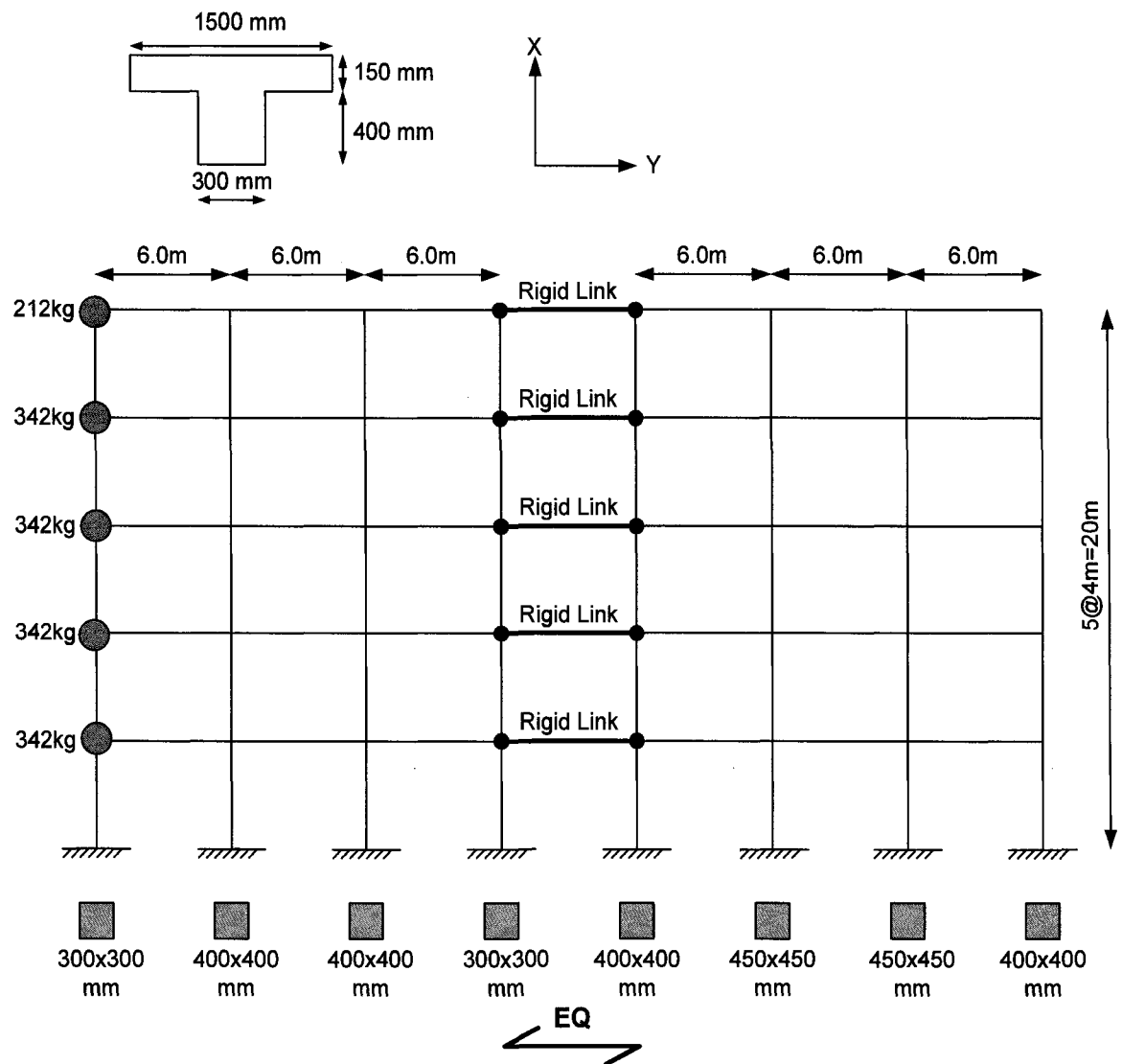


Figure 2.21 Five storey plane frame

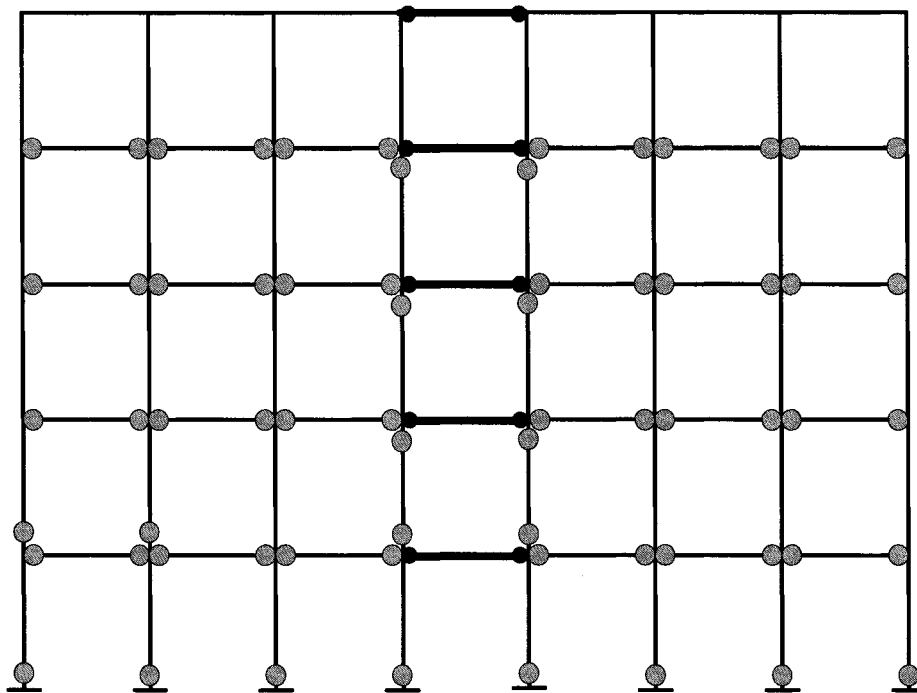


Figure 2.22 Nodes where nonlinear deformation occur

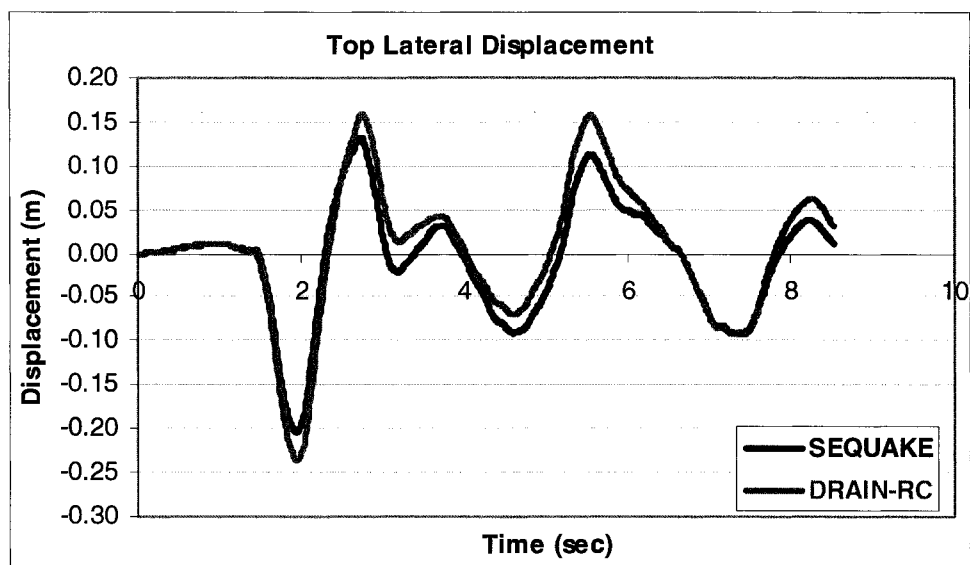


Figure 2.23 Top lateral displacement response

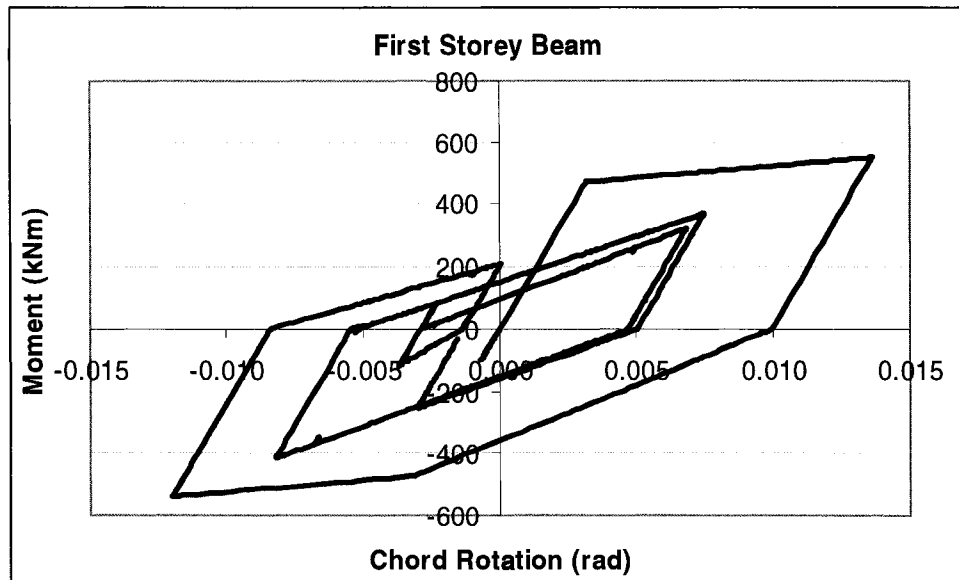


Figure 2.24 Flexural moment-chord rotation relationship of a first storey beam

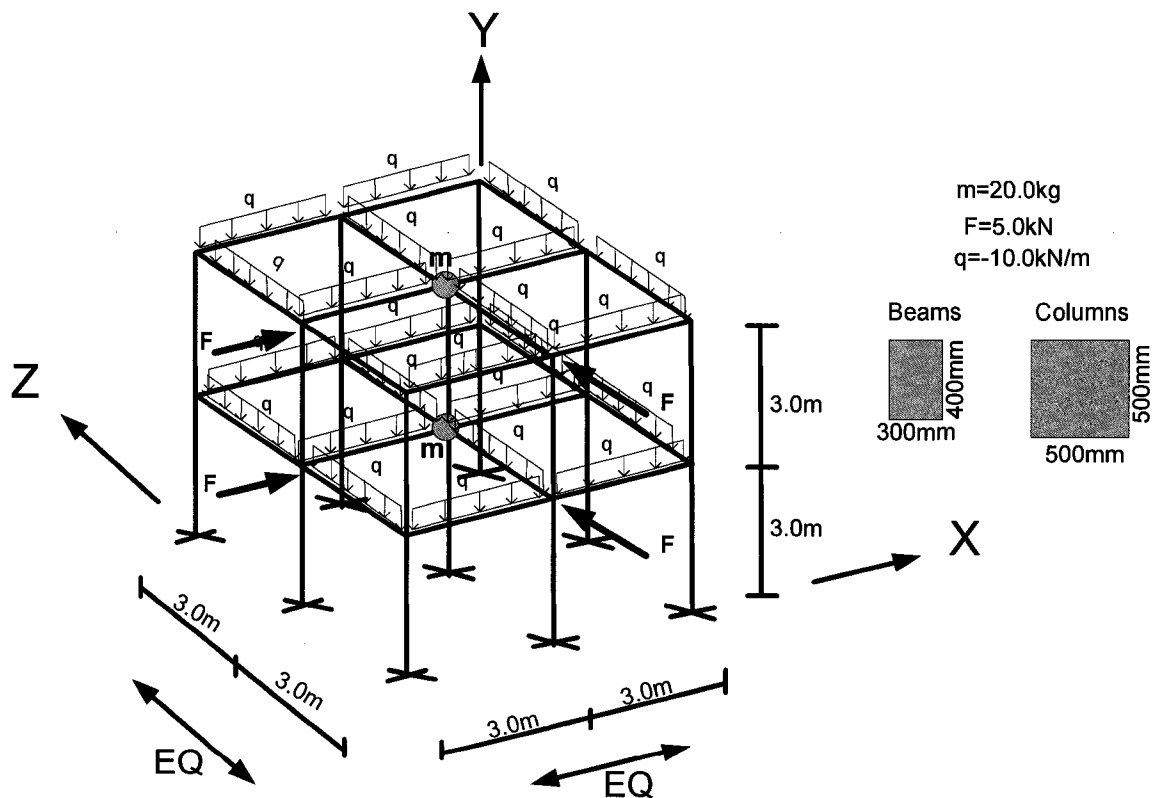


Figure 2.25 Three-dimensional structure example

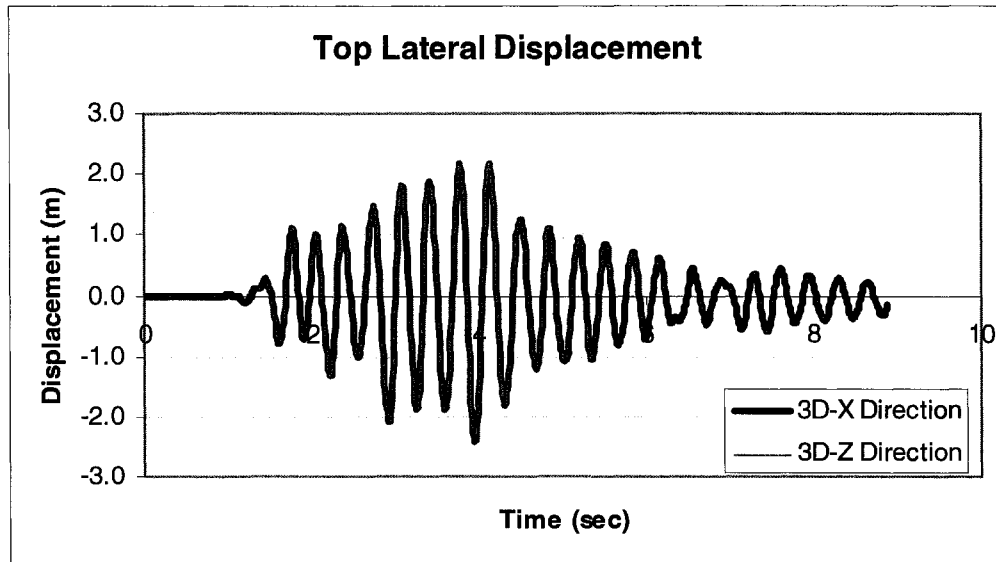


Figure 2.26 Top lateral displacement response of 3D structure

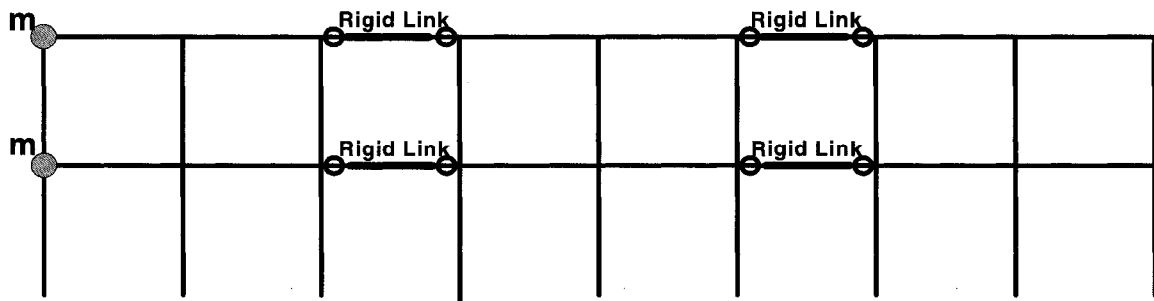


Figure 2.27 Two dimensional model as frames in series

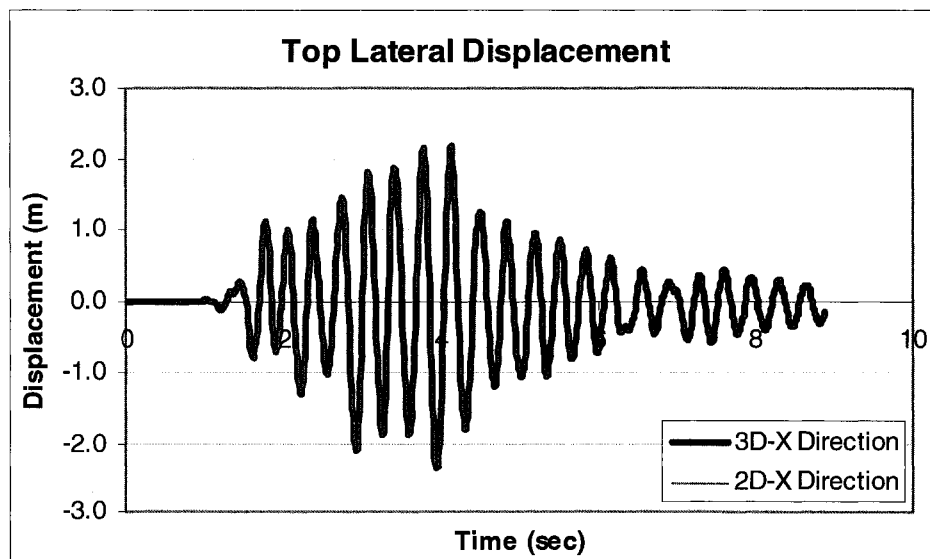


Figure 2.28 Top lateral displacement response of 3D structure and its 2D model

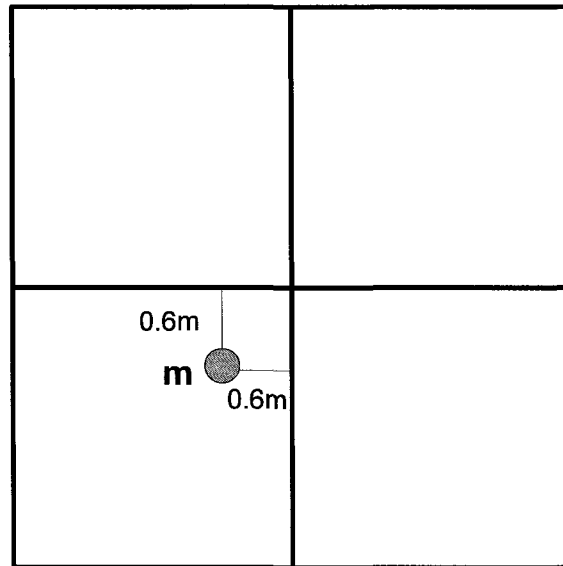


Figure 2.29 Floor plan of 3D structure – mass with eccentricity

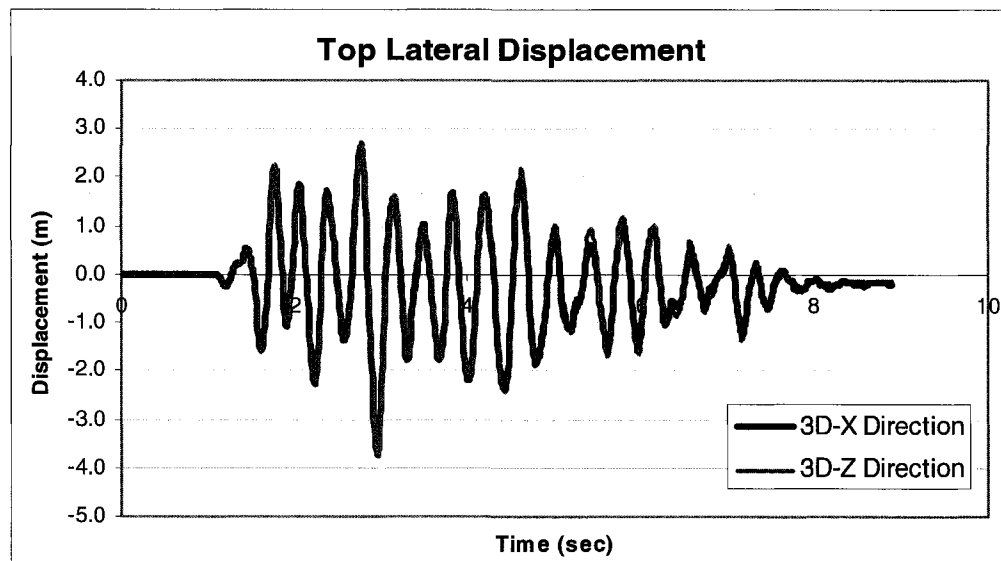


Figure 2.30 Top lateral displacement response of 3D structure - mass with eccentricity

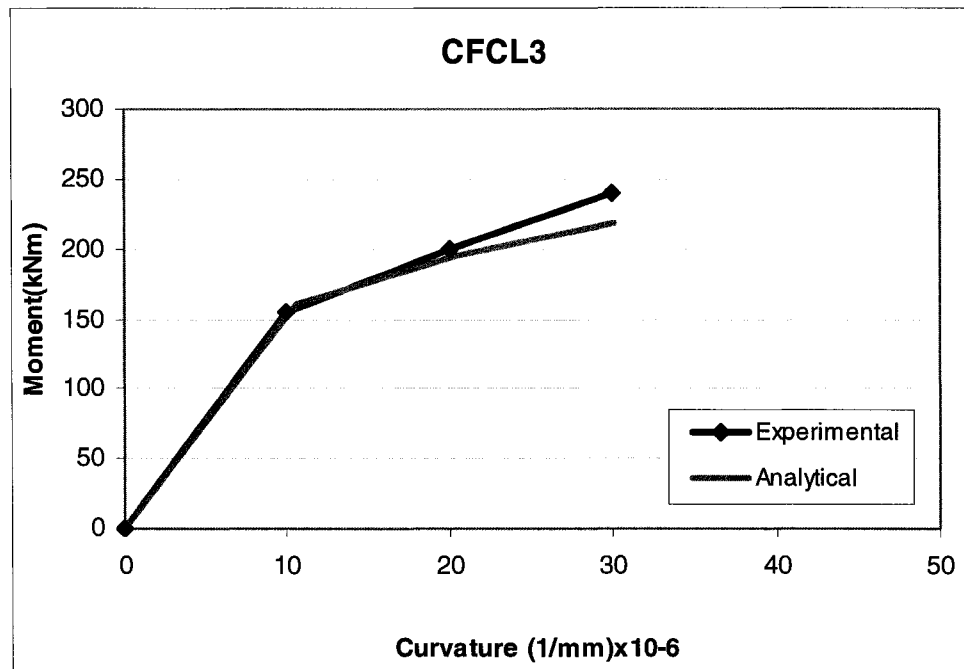


Figure 2.31 Moment-curvature relationship of Column CFCL3

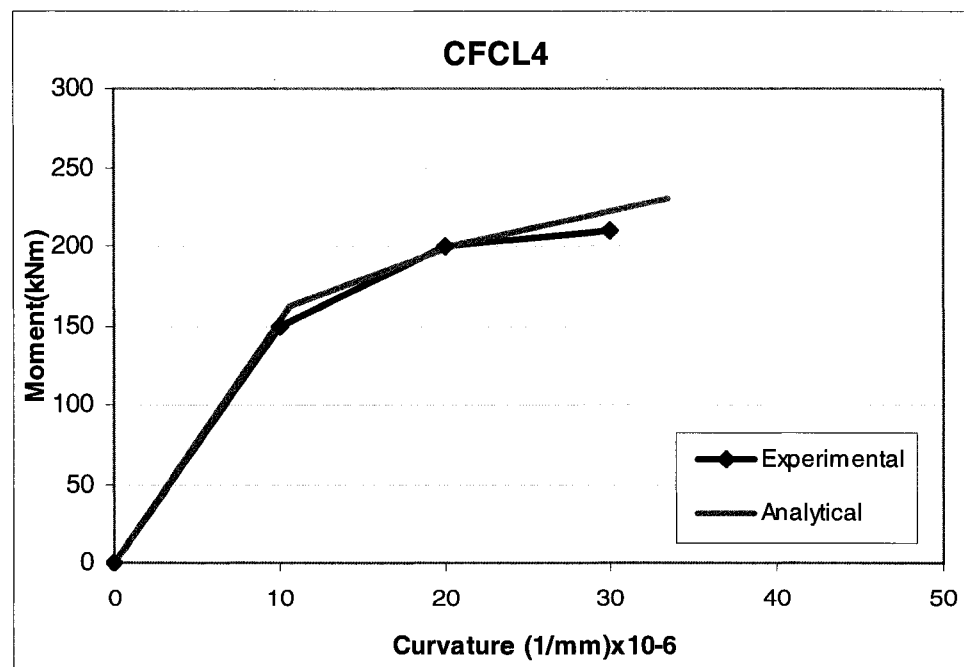


Figure 2.32 Moment-curvature relationship of Column CFCL4

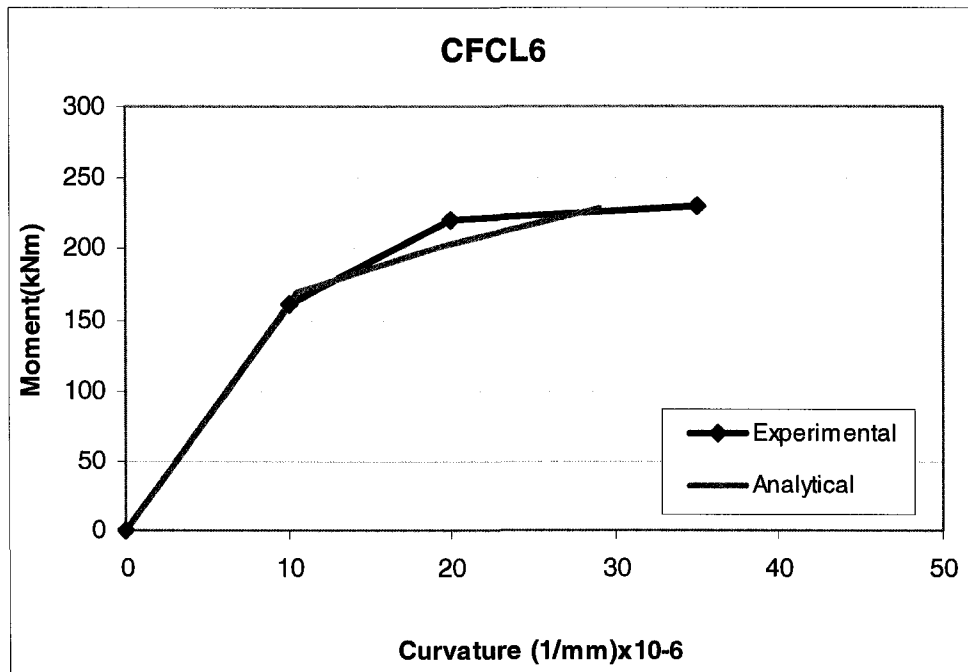


Figure 2.33 Moment-curvature relationship of Column CFCL6

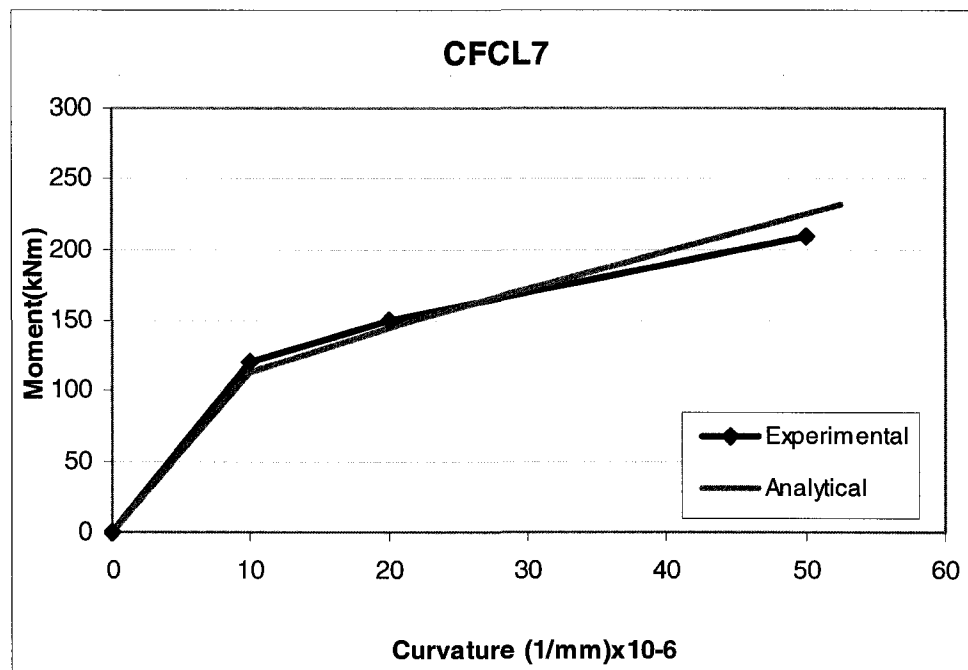


Figure 2.34 Moment-curvature relationship of Column CFCL7

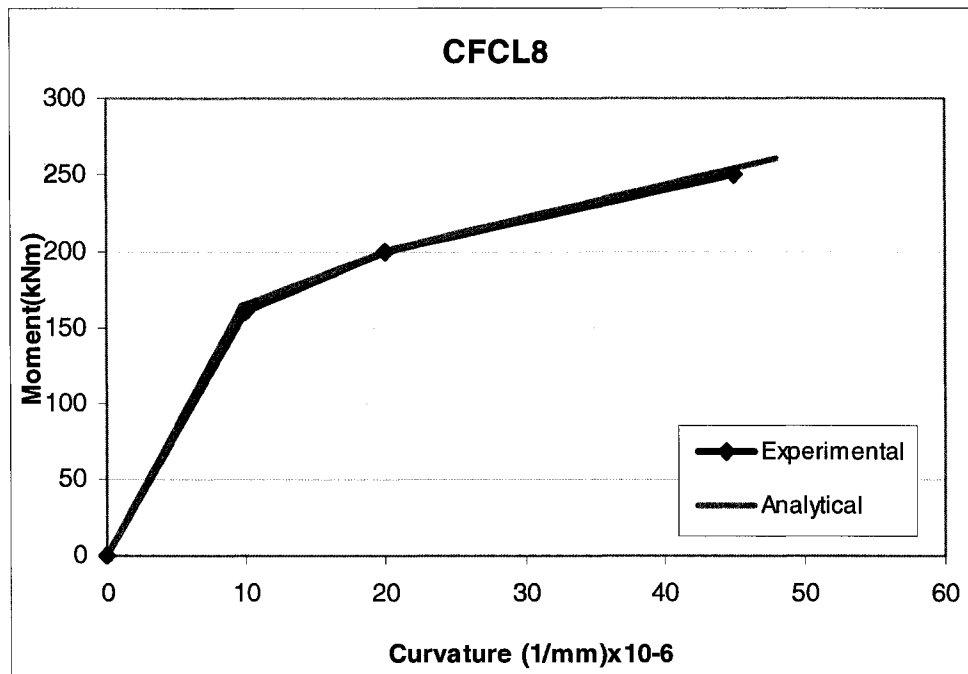


Figure 2.35 Moment-curvature relationship of Column CFCL8

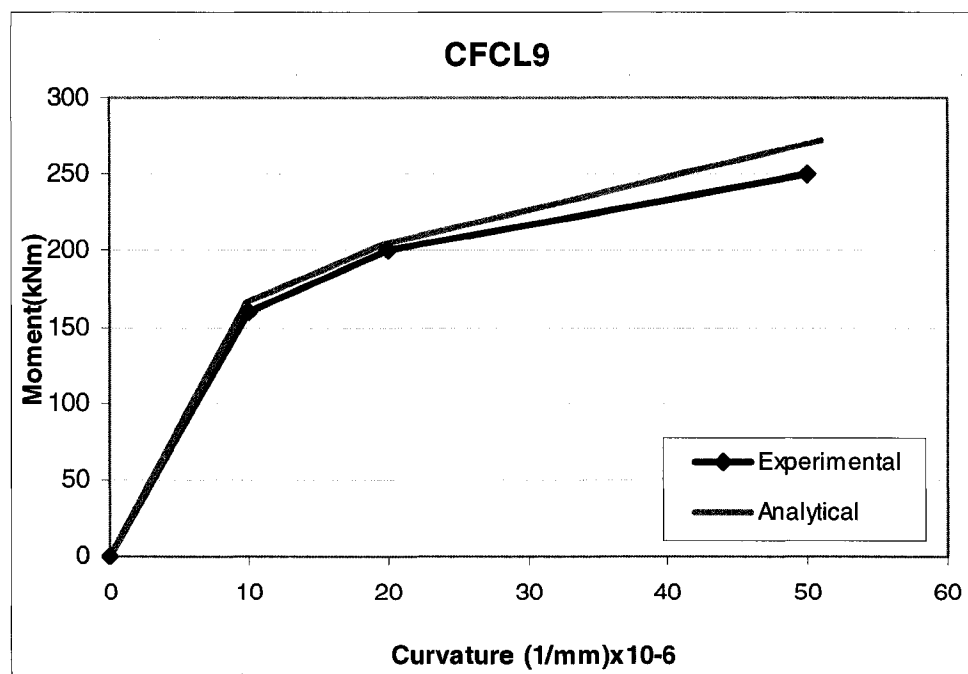


Figure 2.36 Moment-curvature relationship of Column CFCL9

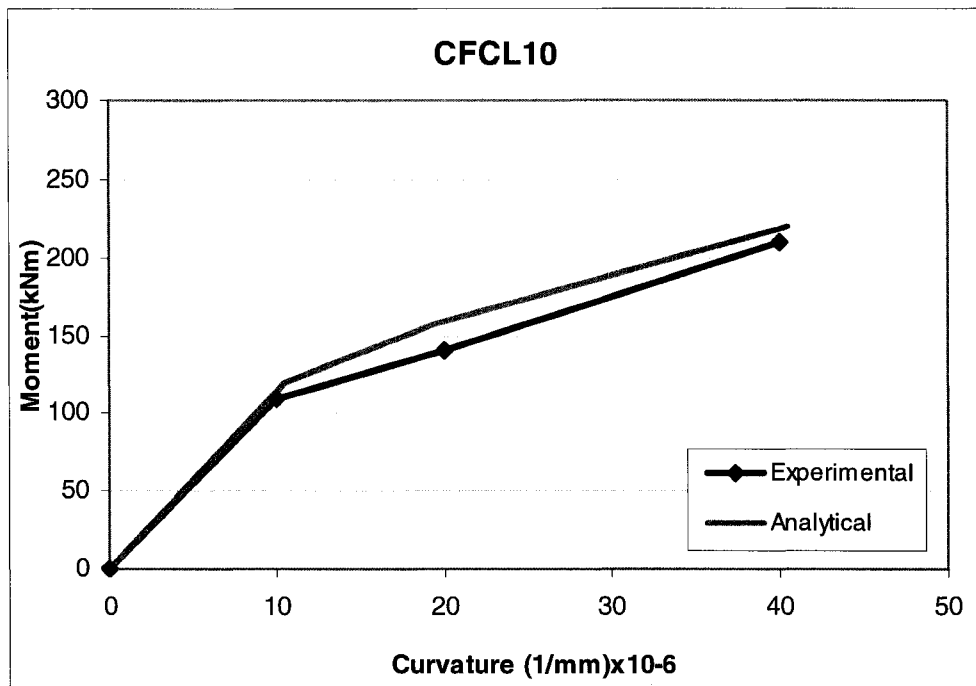


Figure 2.37 Moment-curvature relationship of Column CFCL10

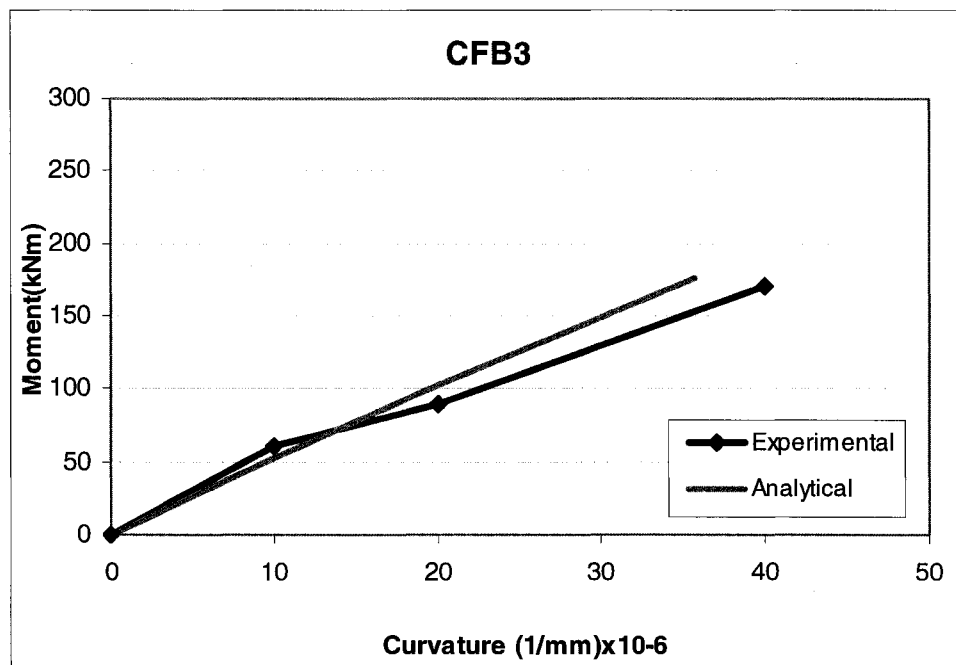


Figure 2.38 Moment-curvature relationship of Beam CFB3

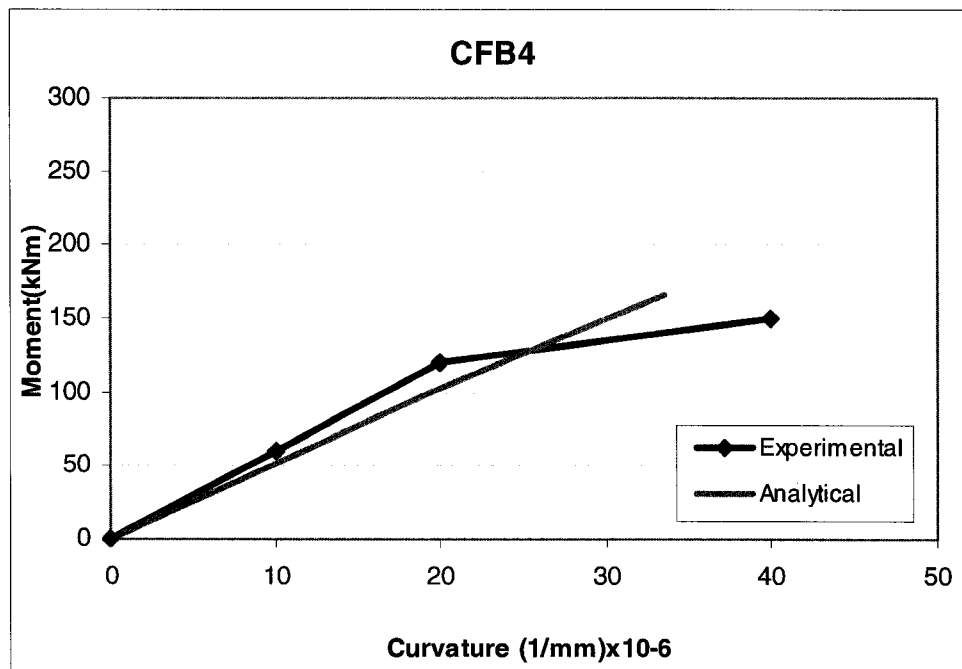


Figure 2.39 Moment-curvature relationship of Beam CFB4

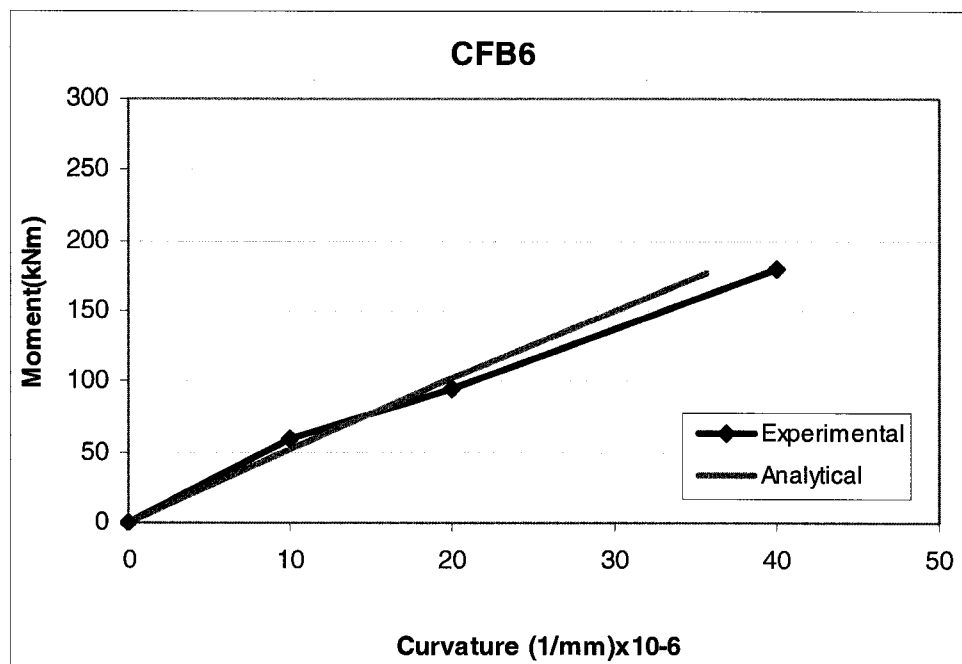


Figure 2.40 Moment-curvature relationship of Beam CFB6

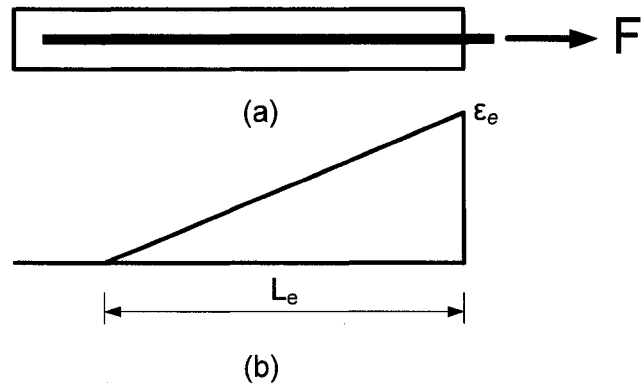


Figure 2.41 FRP reinforcing bar embedded in concrete and its strain distribution

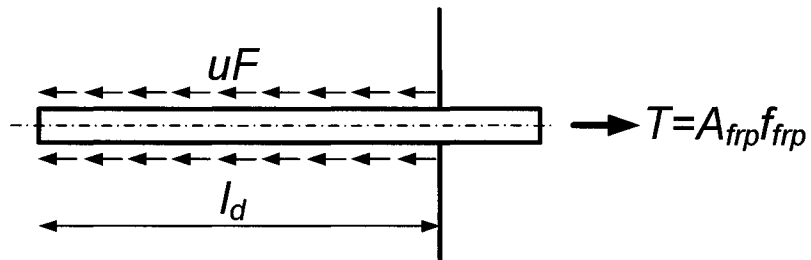


Figure 2.42 Equilibrium of a FRP reinforcing bar embedded in concrete

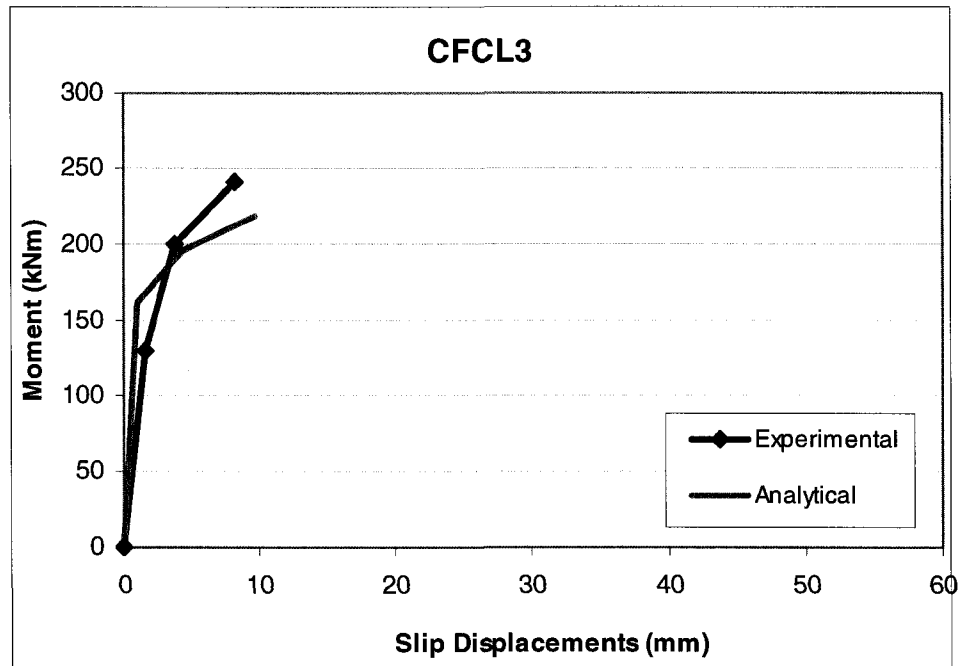


Figure 2.43 Moment-slip displacement relationship of Column CFCL3

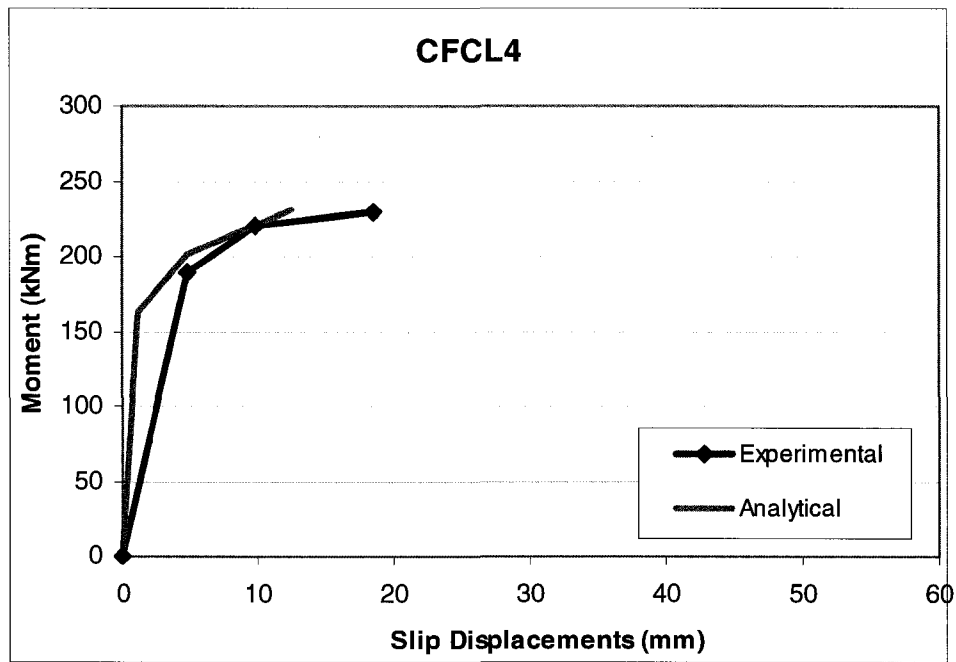


Figure 2.44 Moment-slip displacement relationship of Column CFCL4

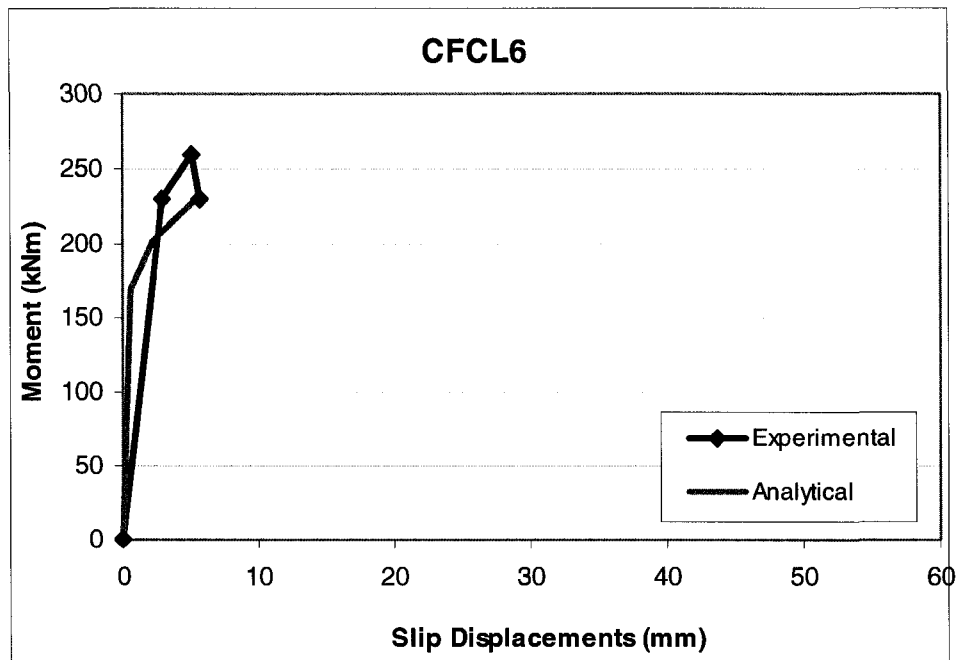


Figure 2.45 Moment-slip displacement relationship of Column CFCL6

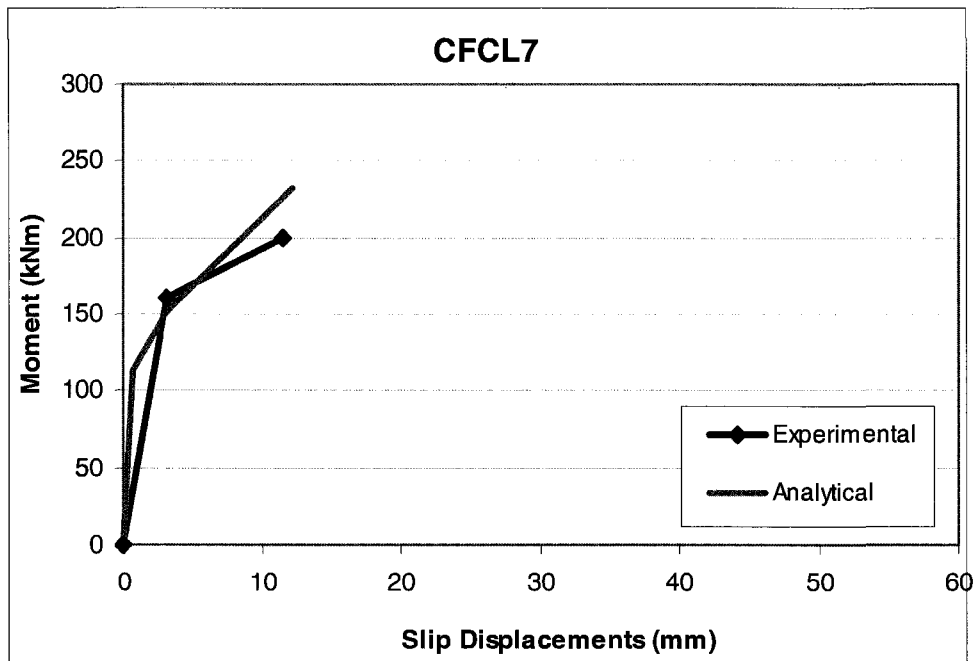


Figure 2.46 Moment-slip displacement relationship of Column CFCL7

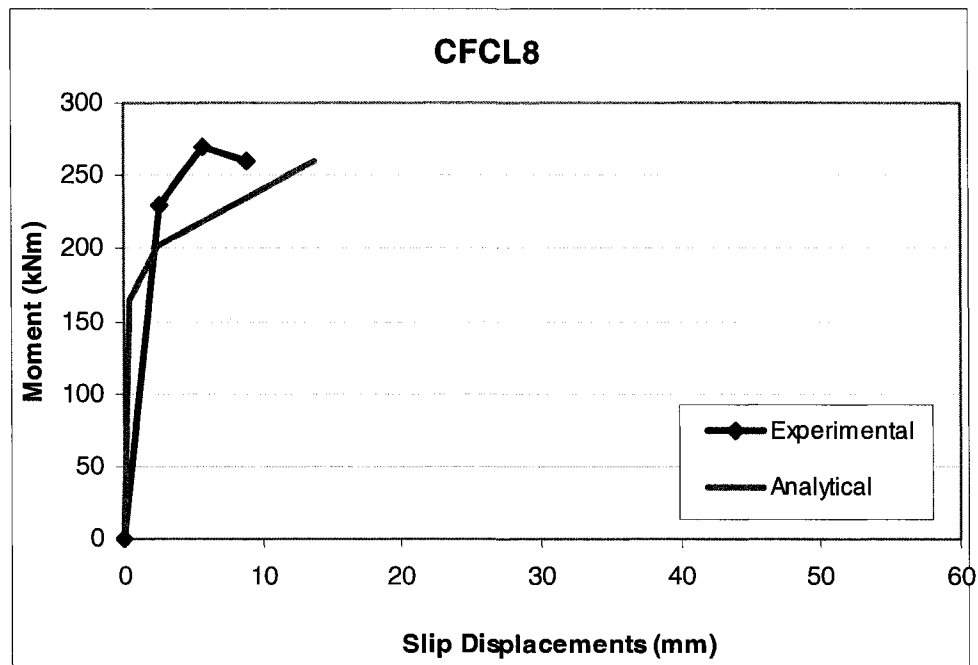


Figure 2.47 Moment-slip displacement relationship of Column CFCL8

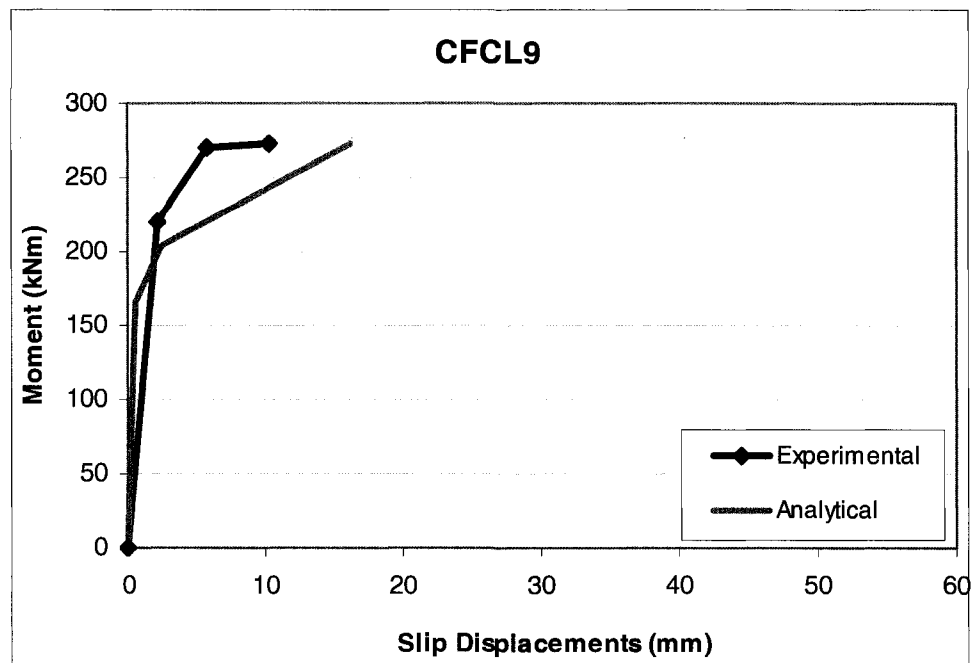


Figure 2.48 Moment-slip displacement relationship of Column CFCL9

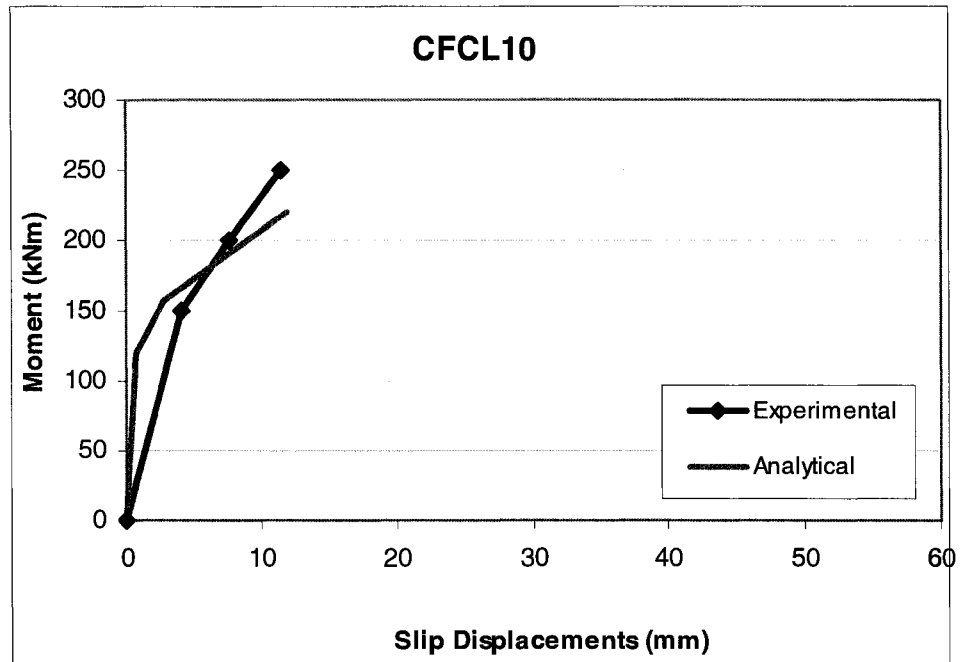


Figure 2.49 Moment-slip displacement relationship of Column CFCL10

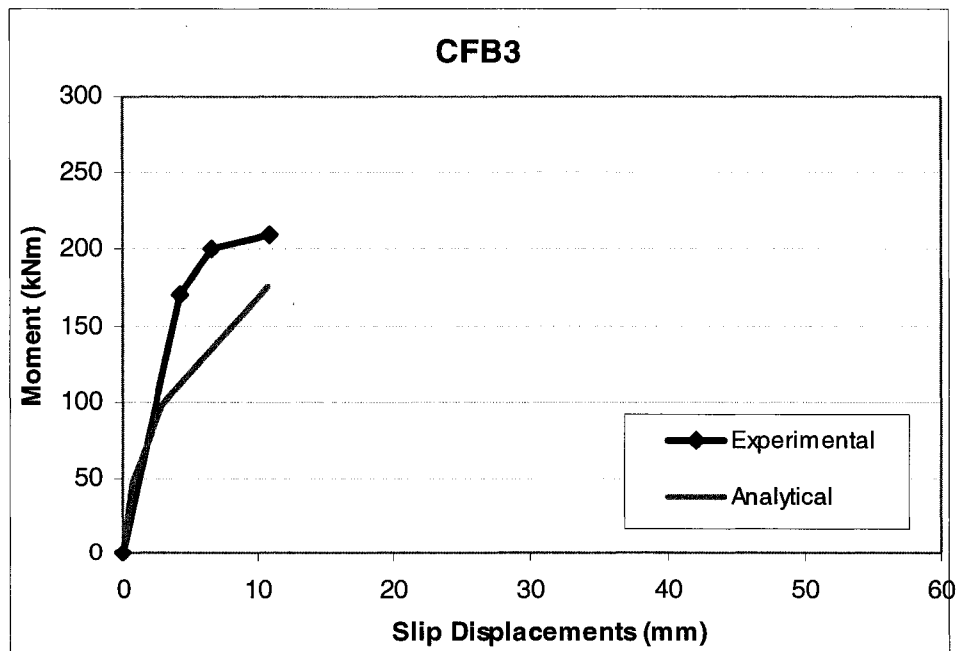


Figure 2.50 Moment-slip displacement relationship of Beam CFB3

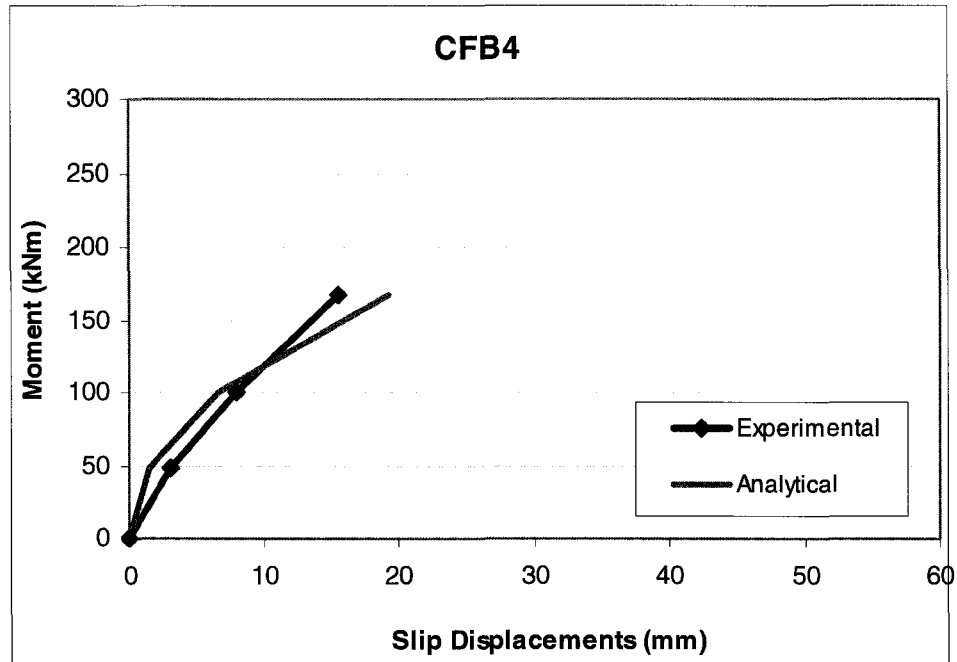


Figure 2.51 Moment-slip displacement relationship of Beam CFB4

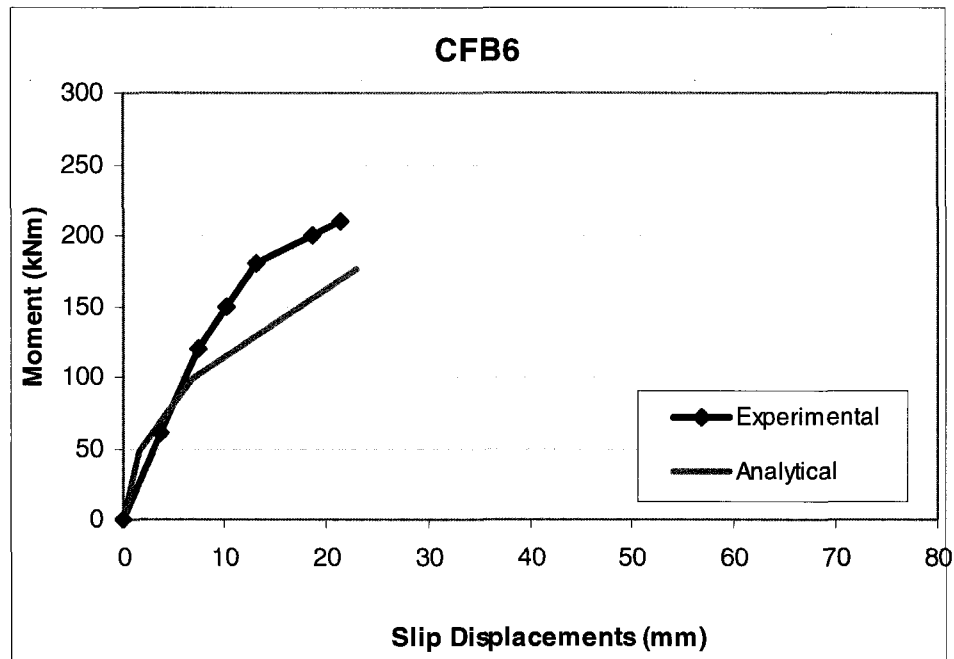


Figure 2.52 Moment-slip displacement relationship of Beam CFB6

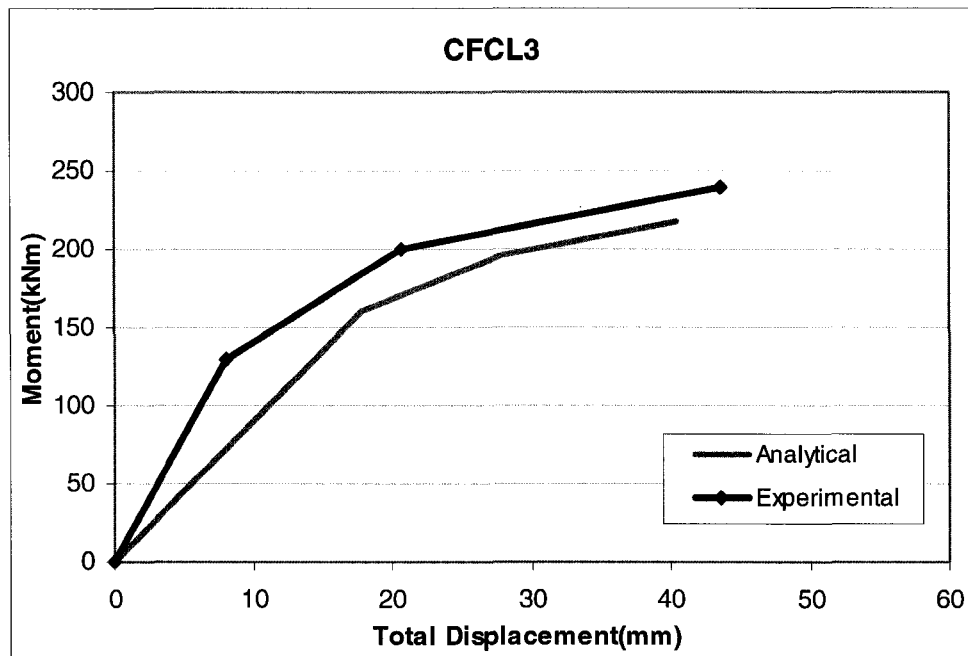


Figure 2.53 Moment-total displacement relationship of Column CFCL3

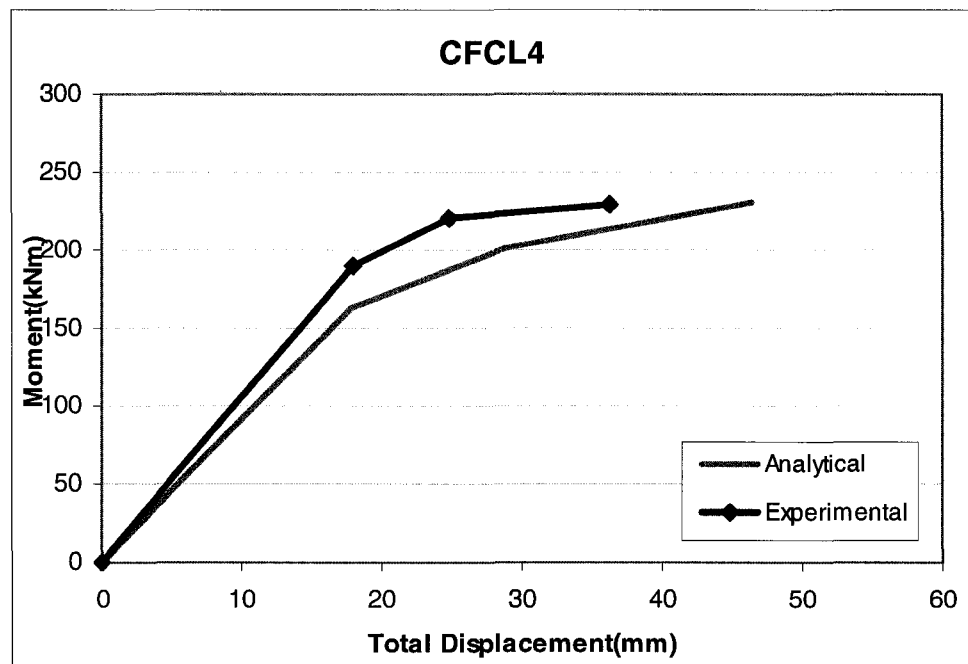


Figure 2.54 Moment-total displacement relationship of Column CFCL4

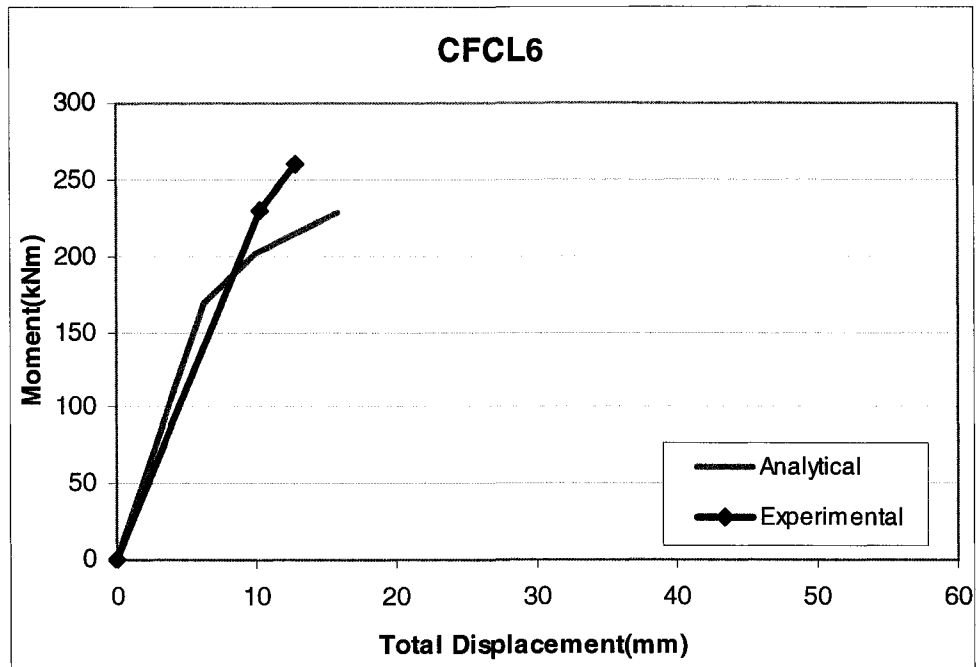


Figure 2.55 Moment-total displacement relationship of Column CFCL6

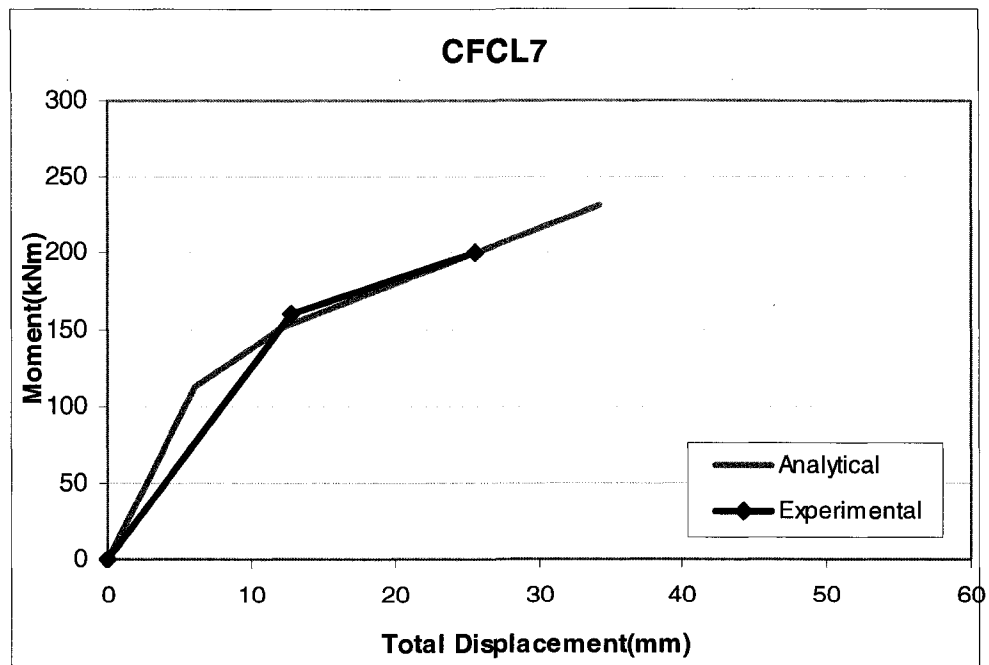


Figure 2.56 Moment-total displacement relationship of Column CFCL7

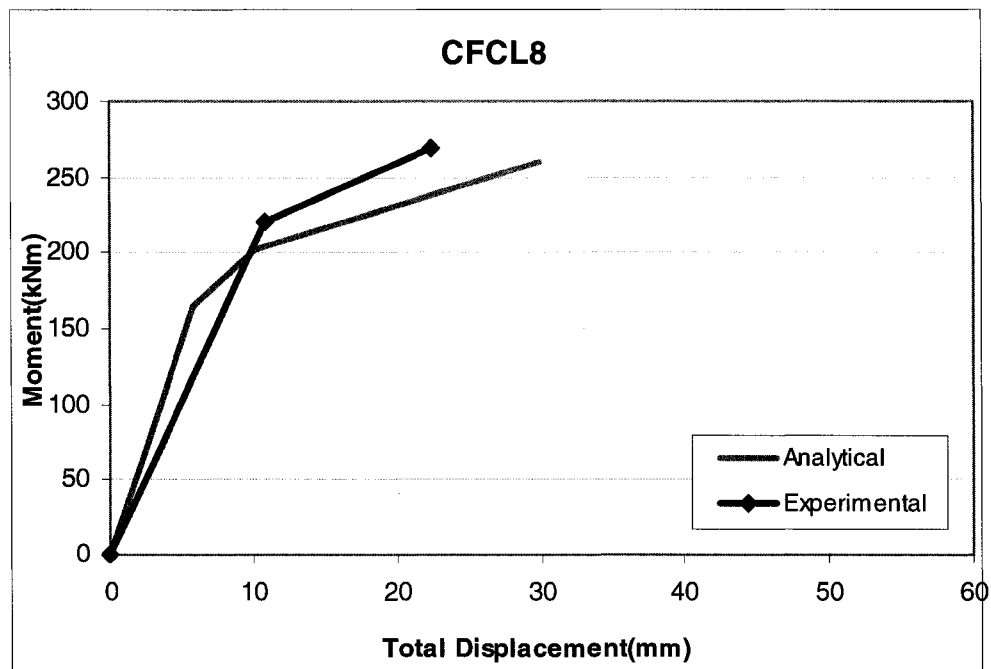


Figure 2.57 Moment-total displacement relationship of Column CFCL8

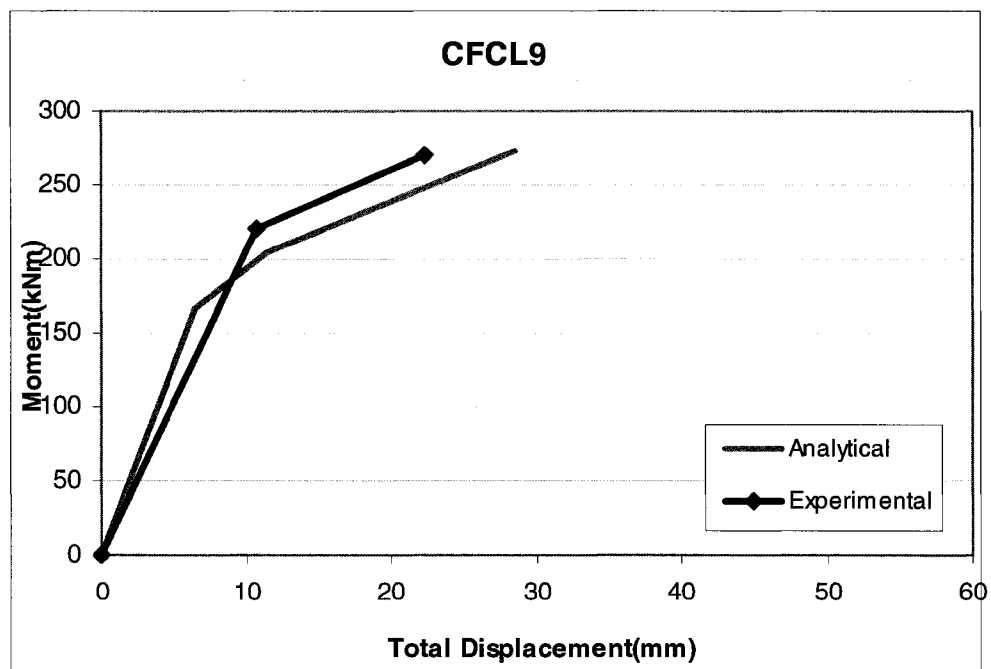


Figure 2.58 Moment-total displacement relationship of Column CFCL9

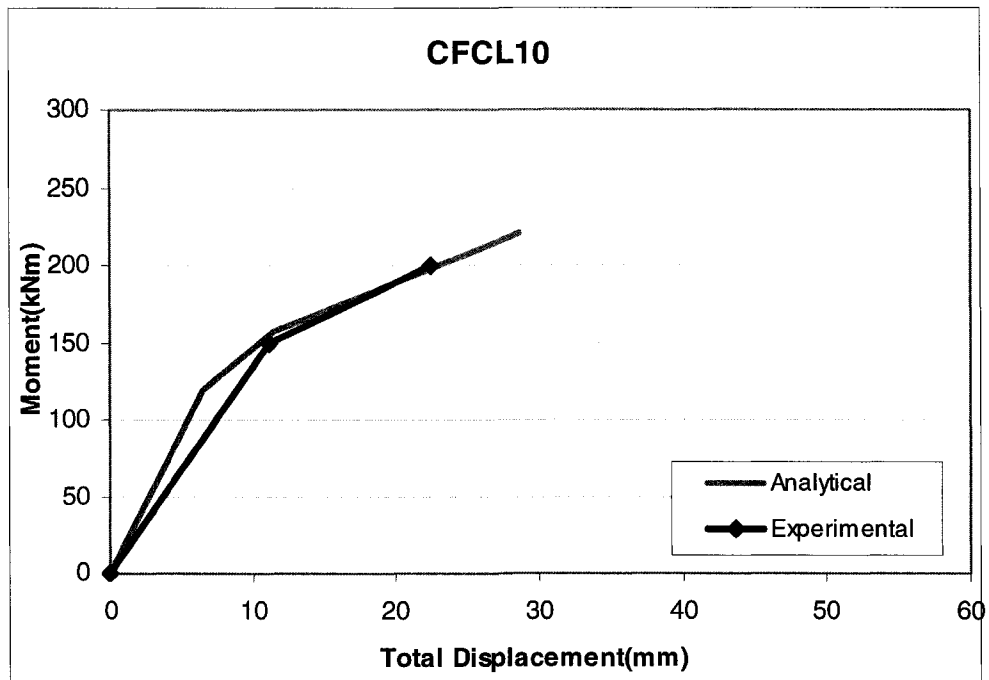


Figure 2.59 Moment-total displacement relationship of Column CFCL10

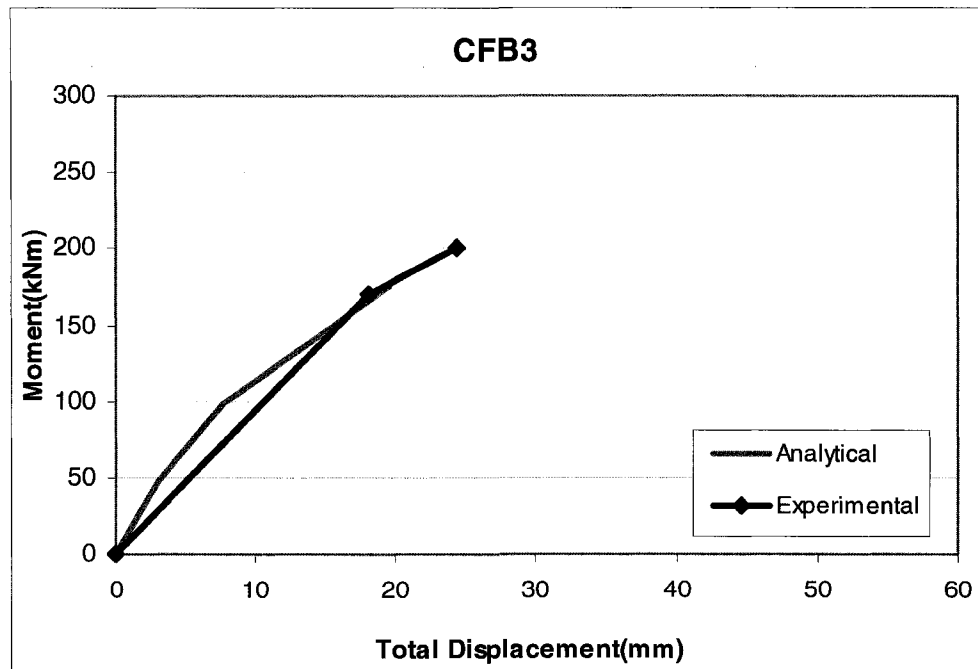


Figure 2.60 Moment-total displacement relationship of Beam CFB3

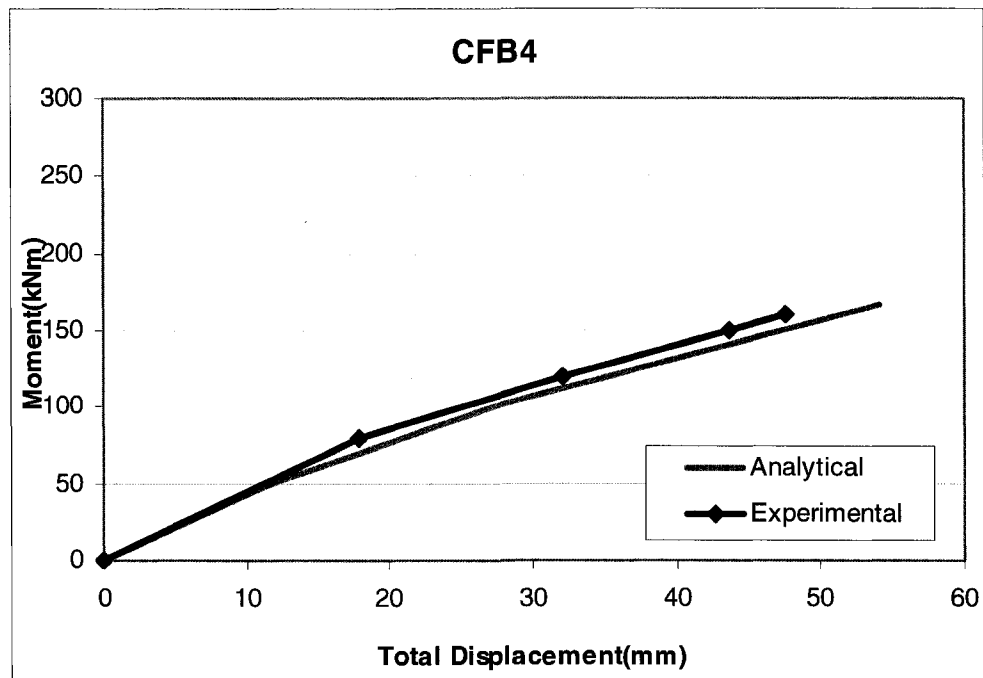


Figure 2.61 Moment-total displacement relationship of Beam CFB4

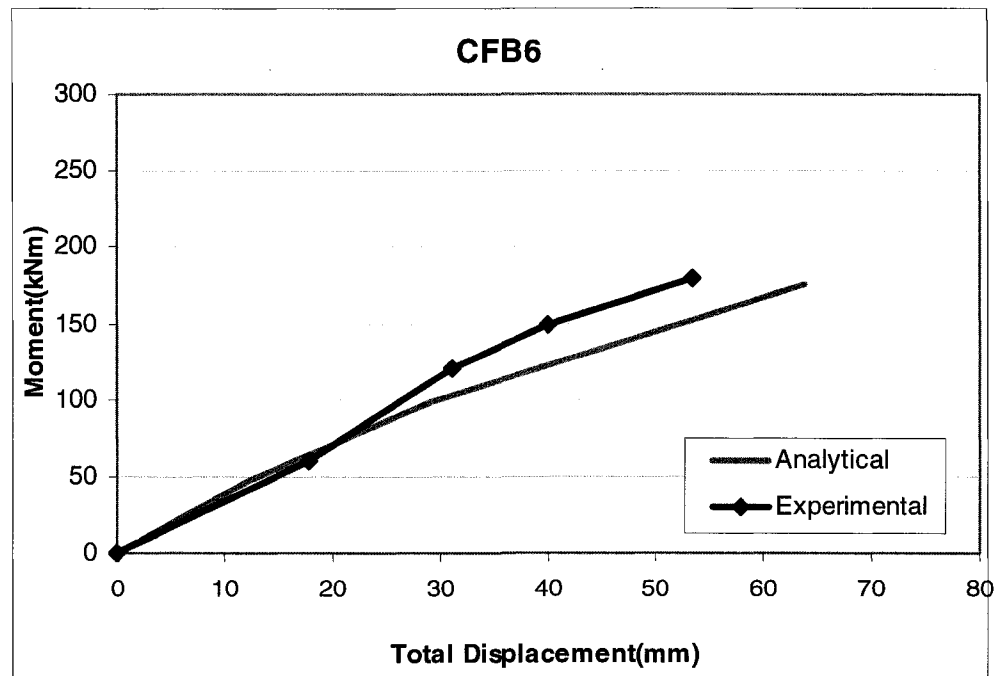


Figure 2.62 Moment-total displacement relationship of Beam CFB6

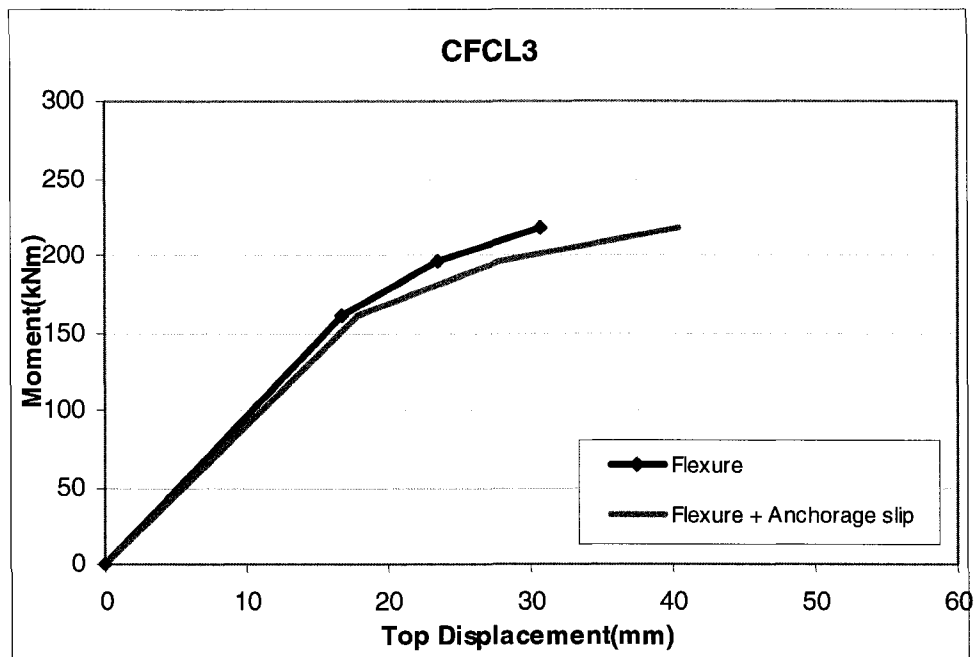


Figure 2.63 Effect of anchorage slip on Column CFCL3

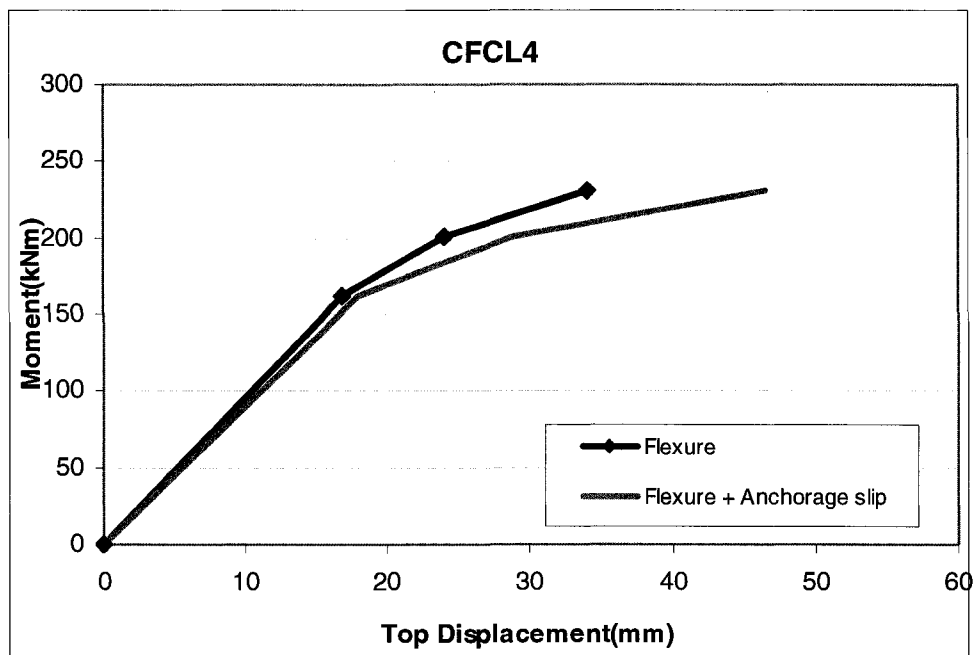


Figure 2.64 Effect of anchorage slip on Column CFCL4

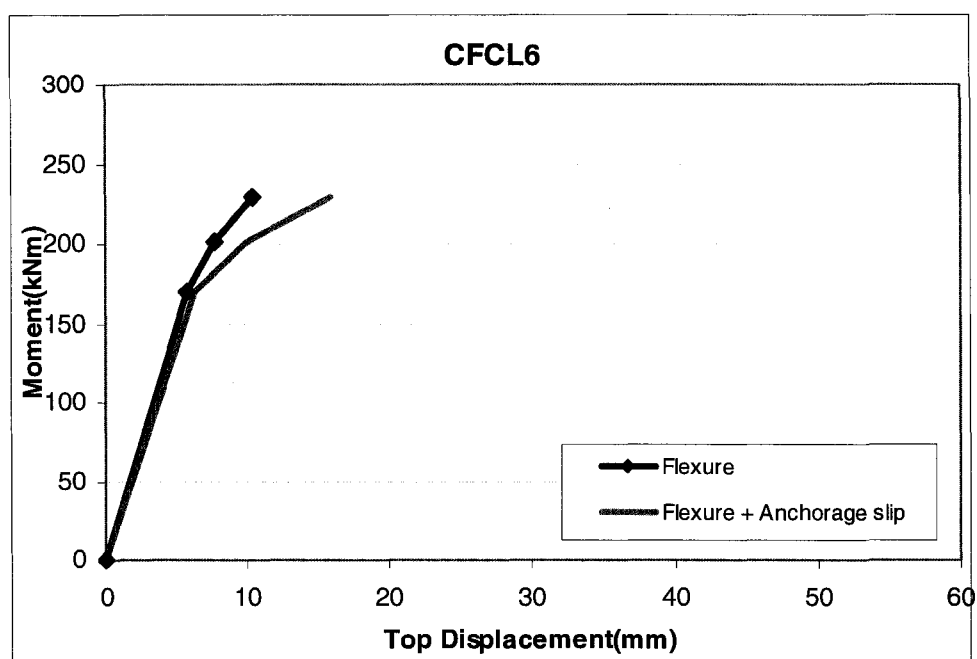


Figure 2.65 Effect of anchorage slip on Column CFCL6

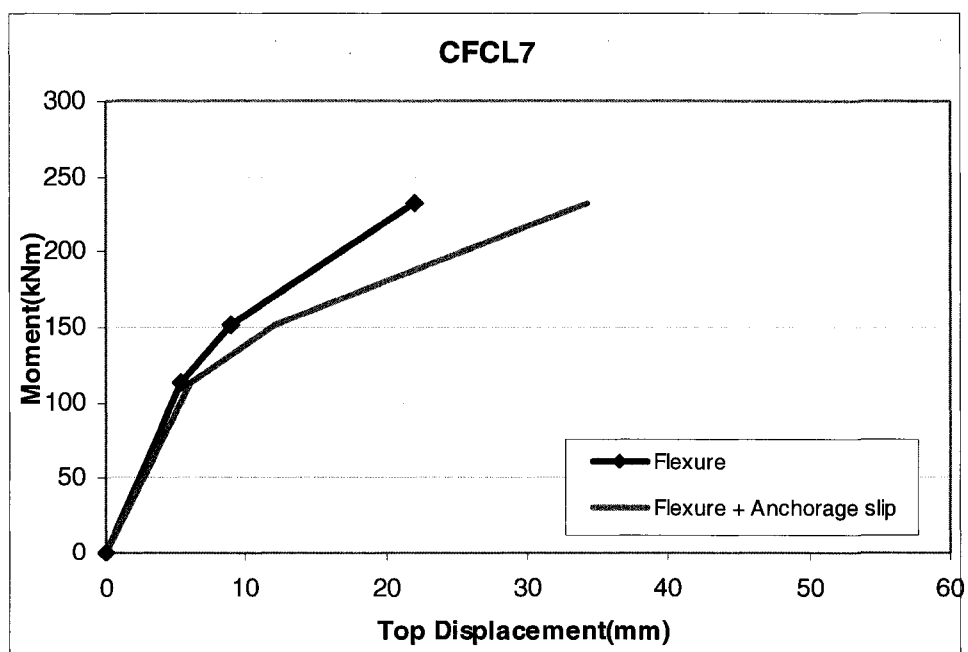


Figure 2.66 Effect of anchorage slip on Column CFCL7

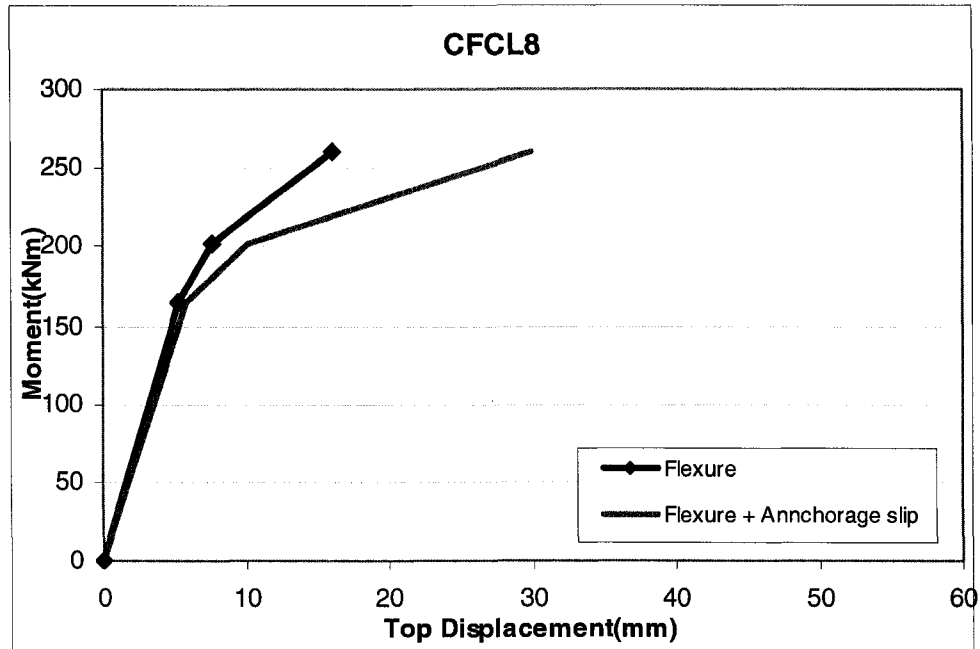


Figure 2.67 Effect of anchorage slip on Column CFCL8

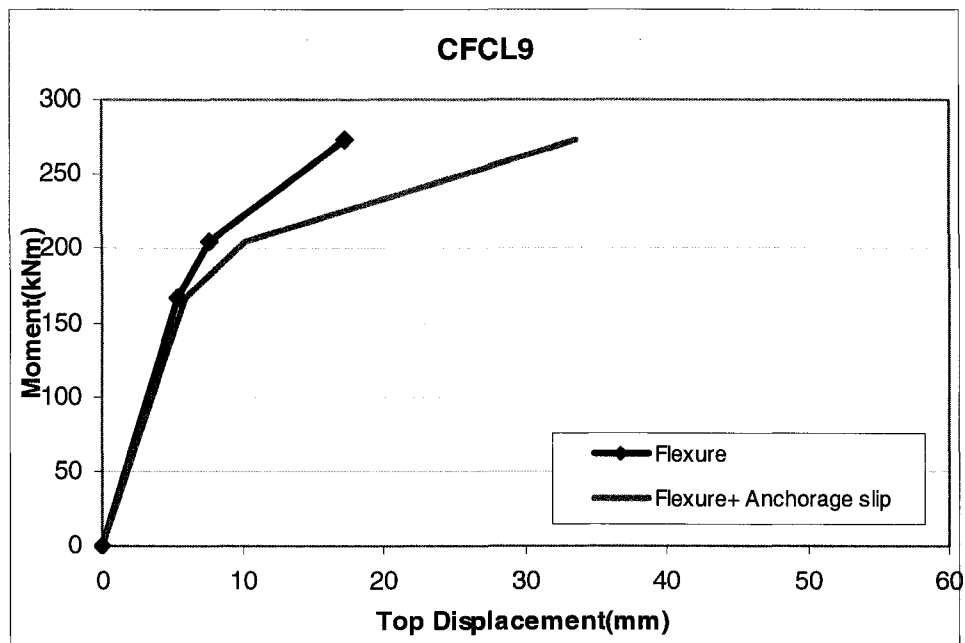


Figure 2.68 Effect of anchorage slip on Column CFCL9

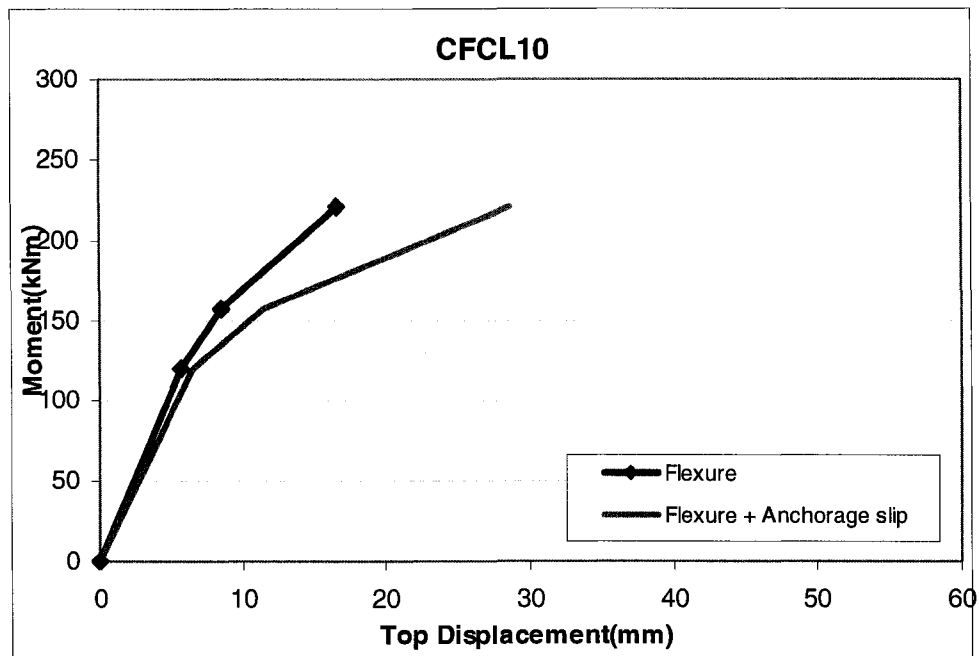


Figure 2.69 Effect of anchorage slip on Column CFCL10

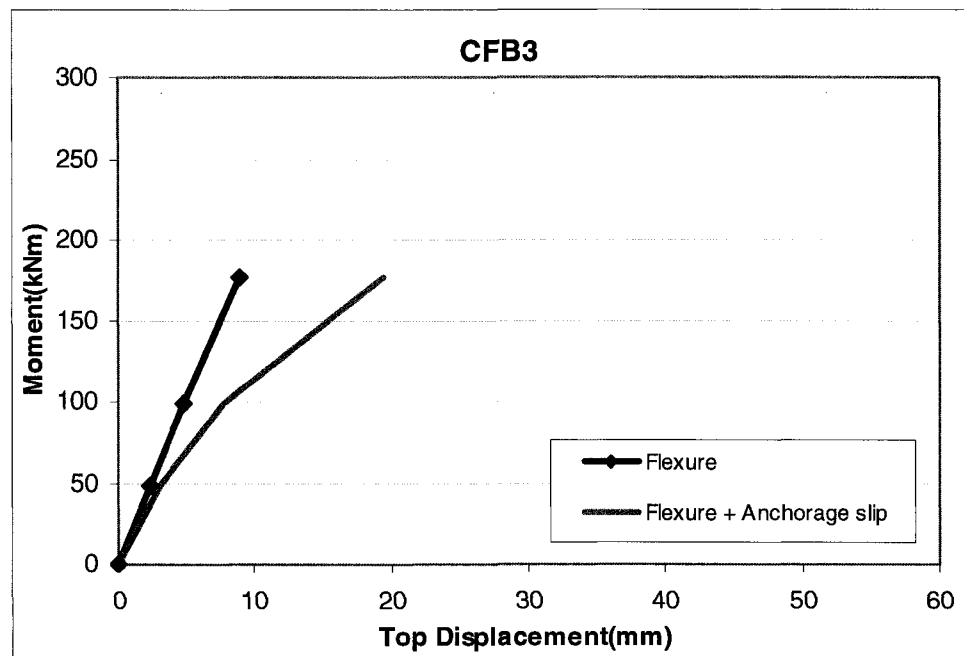


Figure 2.70 Effect of anchorage slip on Beam CFB3

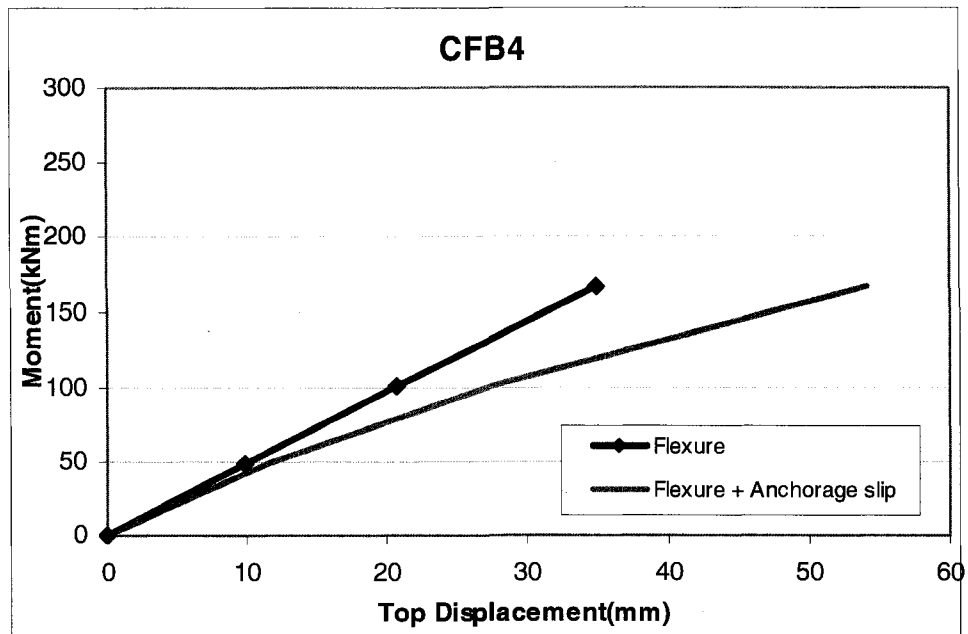


Figure 2.71 Effect of anchorage slip on Beam CFB4

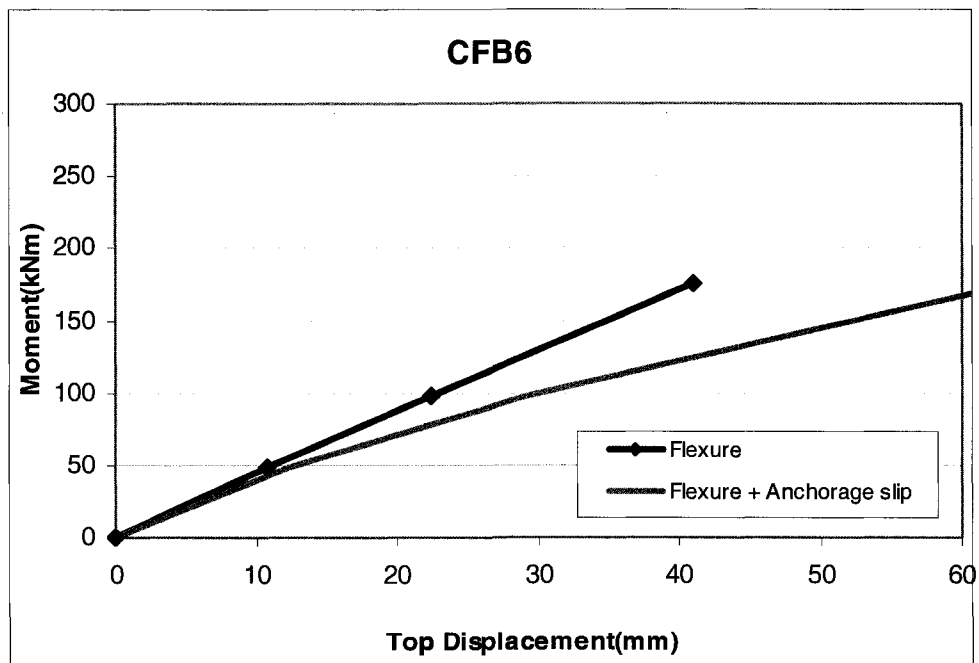


Figure 2.72 Effect of anchorage slip on Beam CFB6

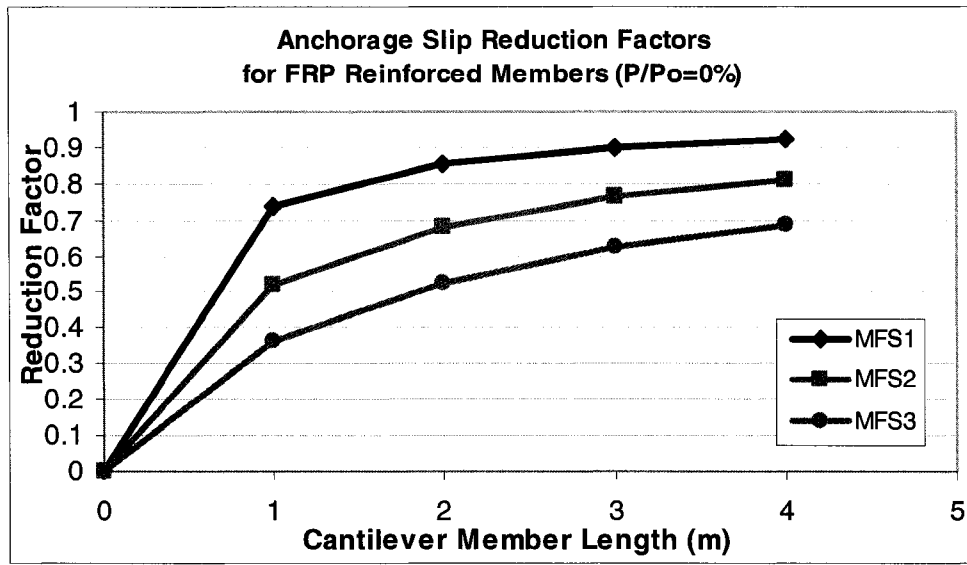


Figure 2.73 Slope reduction factors for FRP reinforced sections with no axial loading

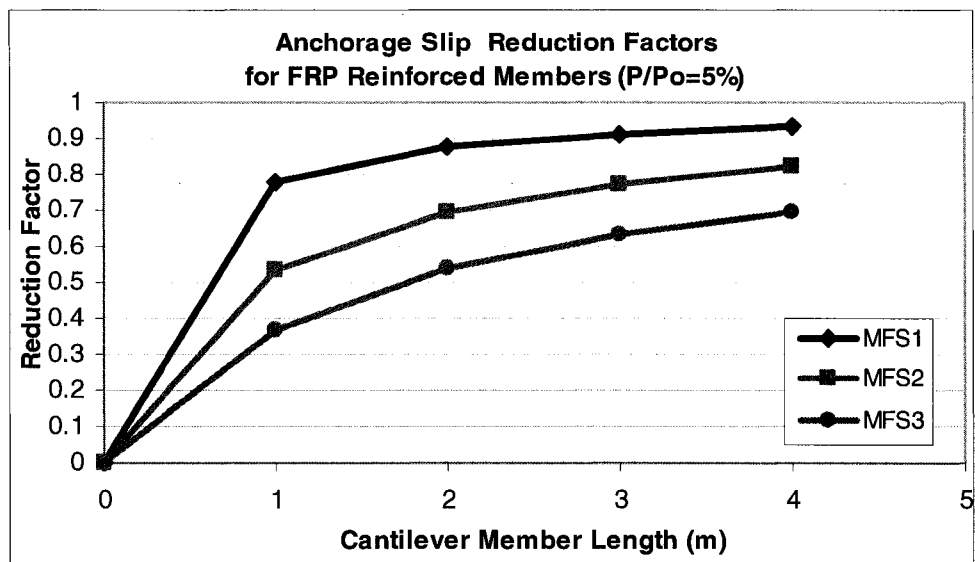


Figure 2.74 Slope reduction factors for FRP reinforced sections with 5% axial loading

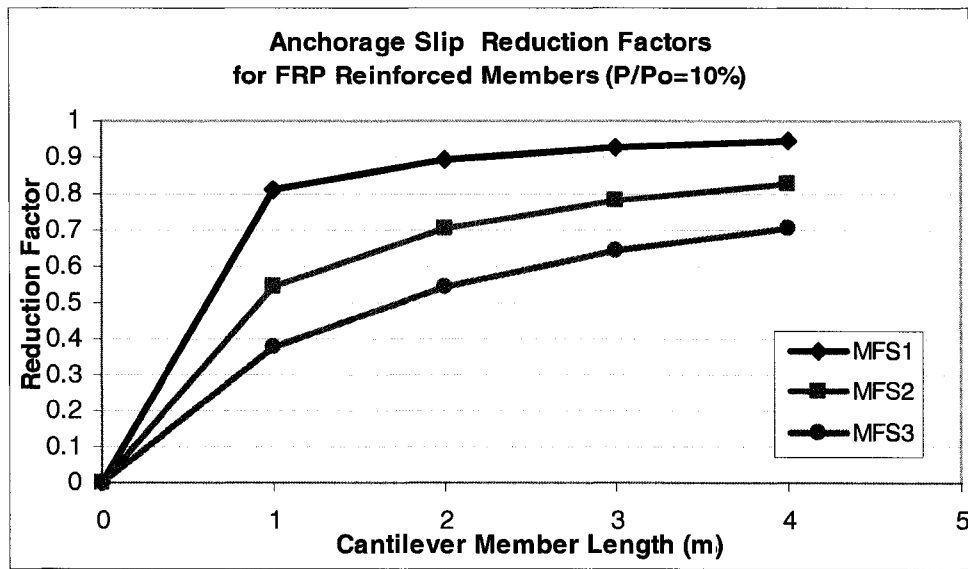


Figure 2.75 Slope reduction factors for FRP reinforced sections with 10% axial loading

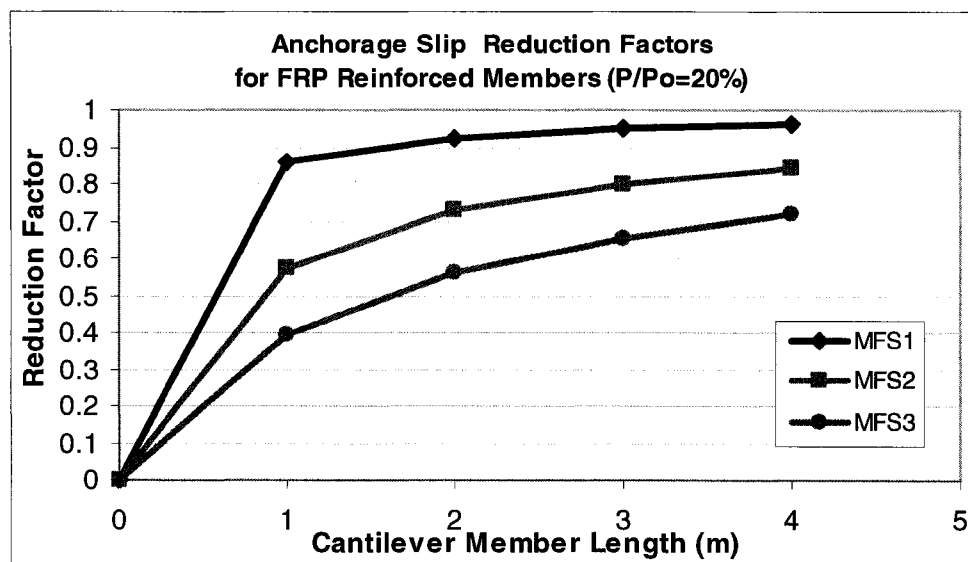


Figure 2.76 Slope reduction factors for FRP reinforced sections with 20% axial loading

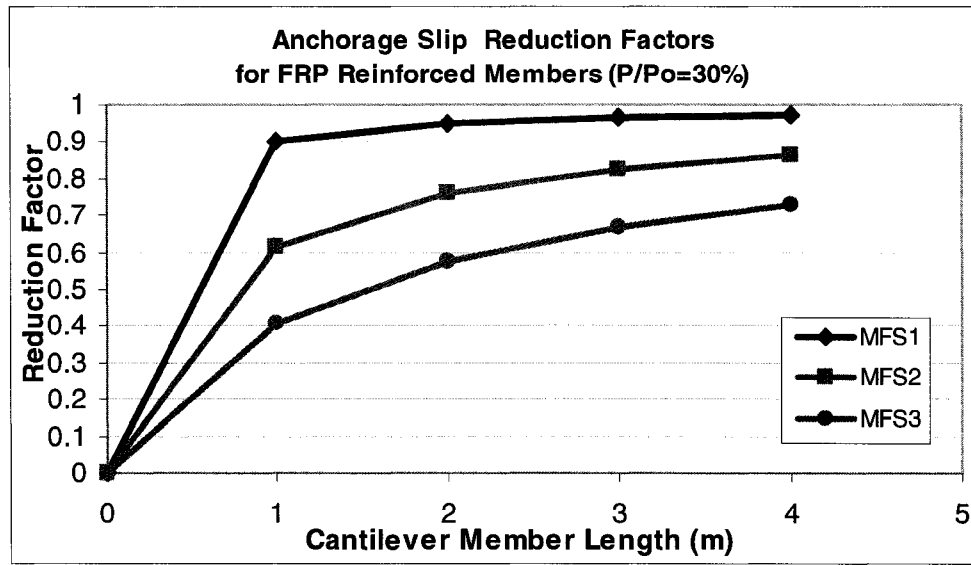


Figure 2.77 Slope reduction factors for FRP reinforced sections with 30% axial loading

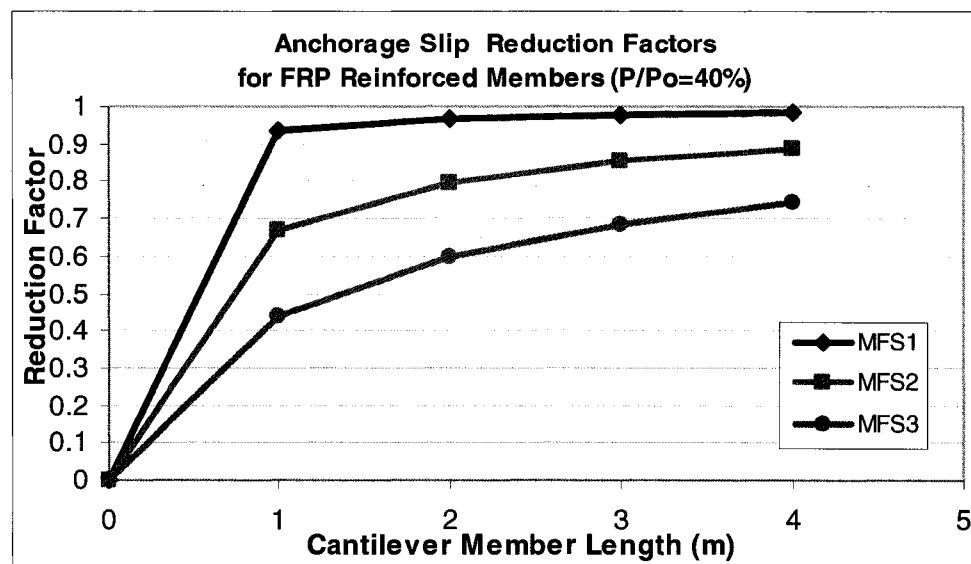


Figure 2.78 Slope reduction factors for FRP reinforced sections with 40% axial loading

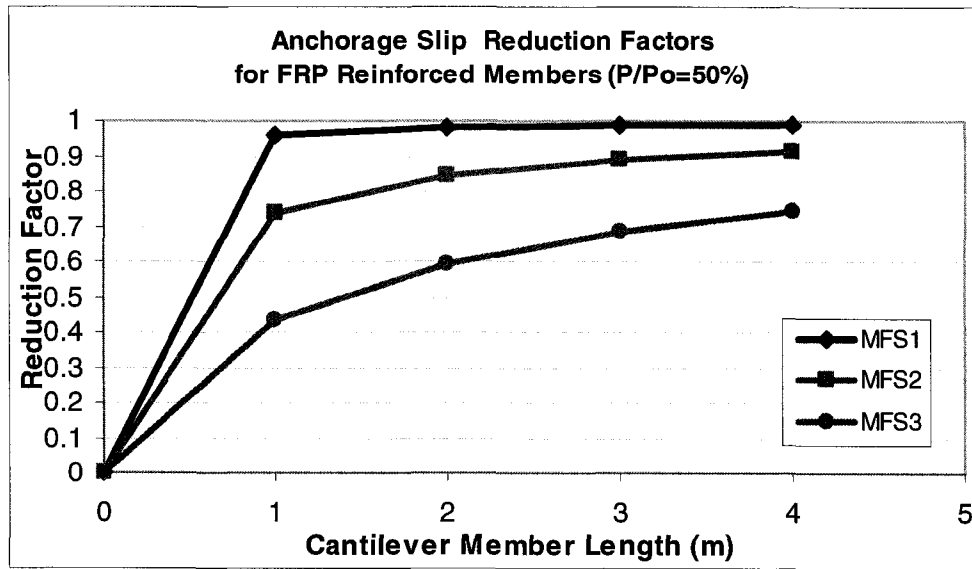


Figure 2.79 Slope reduction factors for FRP reinforced sections with 50% axial loading

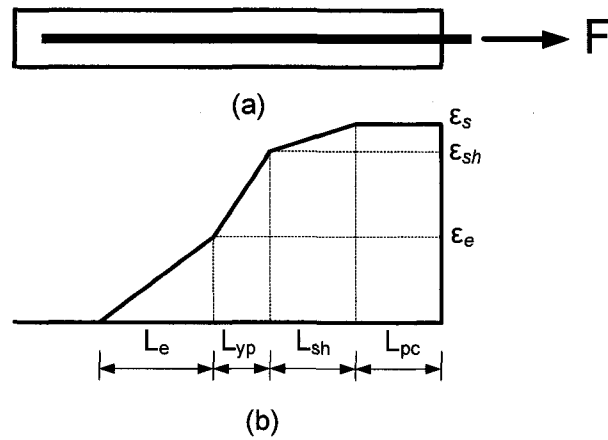


Figure 2.80 Steel reinforcing bar embedded in concrete and its strain distribution
(Alsiwat and Saatcioglu, 1992)

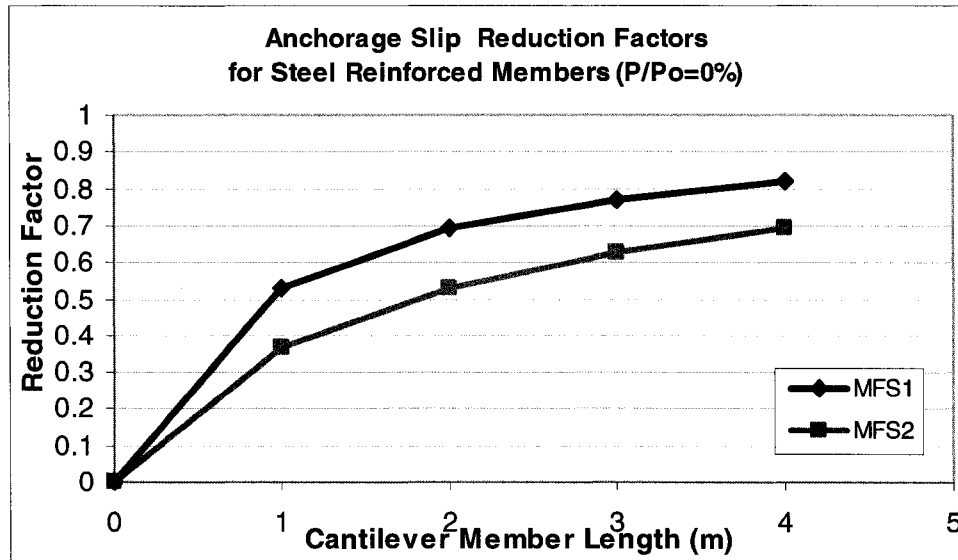


Figure 2.81 Slope reduction factors for steel reinforced sections with no axial loading

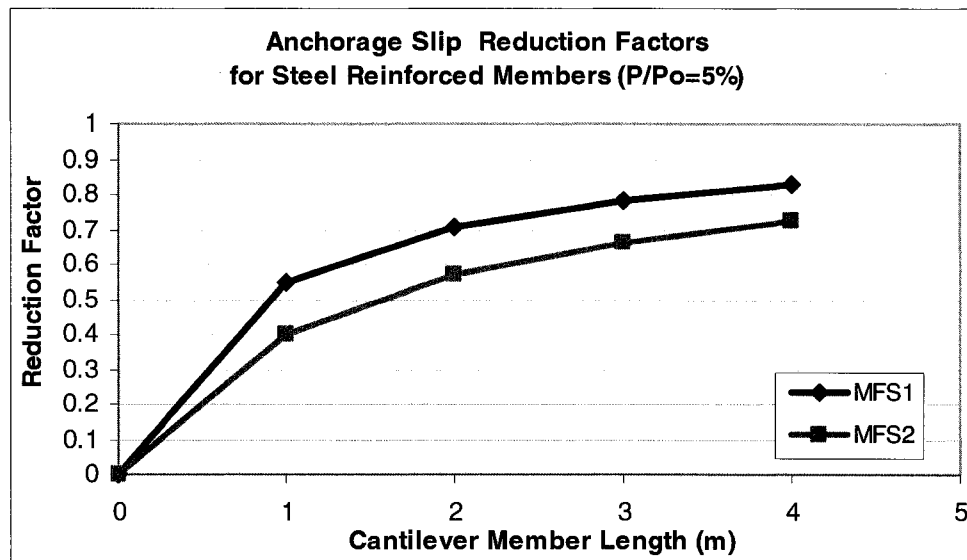


Figure 2.82 Slope reduction factors for steel reinforced sections with 5% axial loading

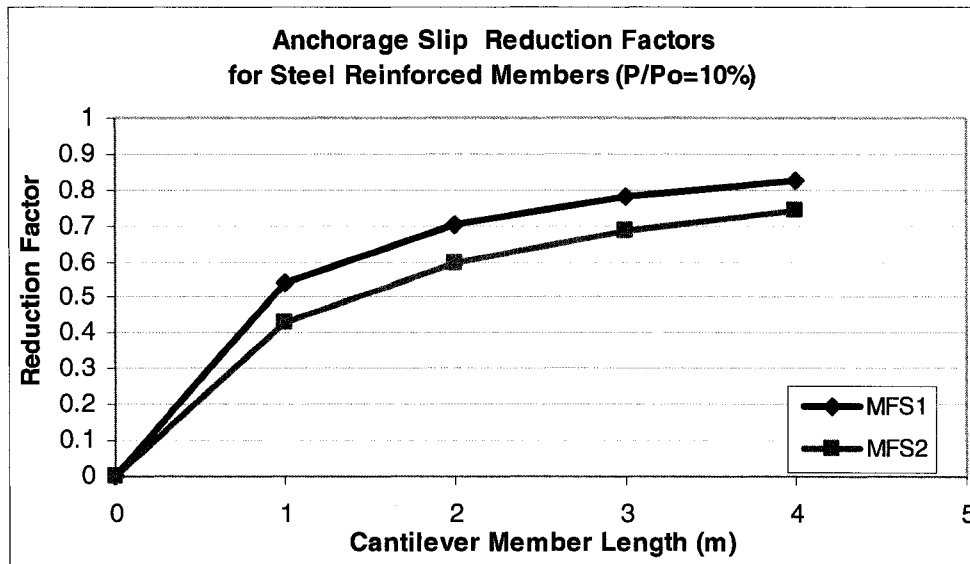


Figure 2.83 Slope reduction factors for steel reinforced sections with 10% axial loading

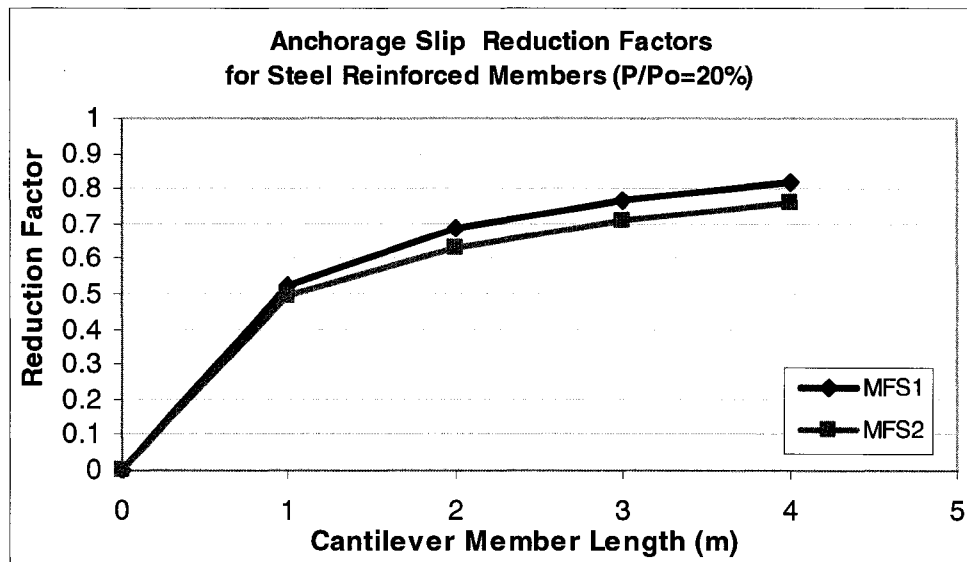


Figure 2.84 Slope reduction factors for steel reinforced sections with 20% axial loading

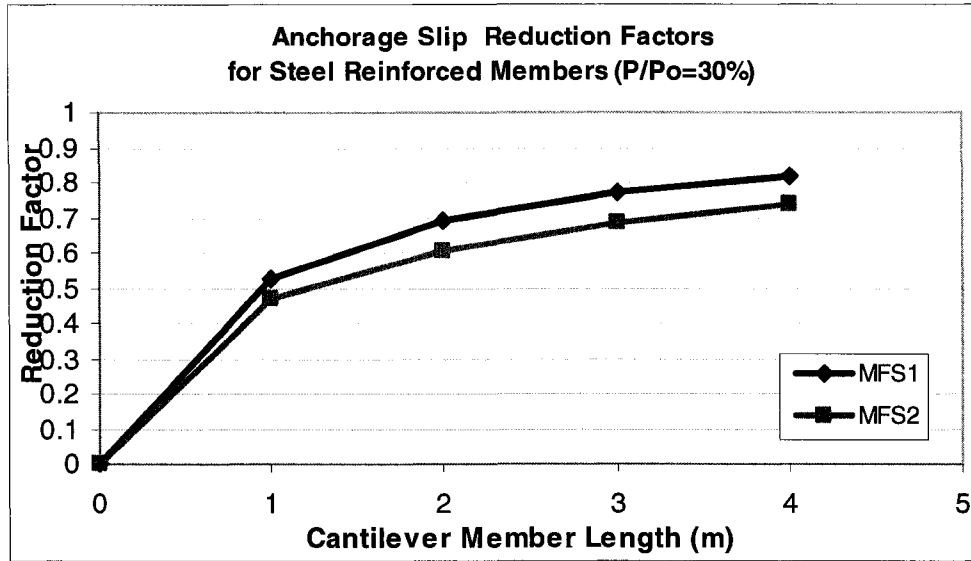


Figure 2.85 Slope reduction factors for steel reinforced sections with 30% axial loading

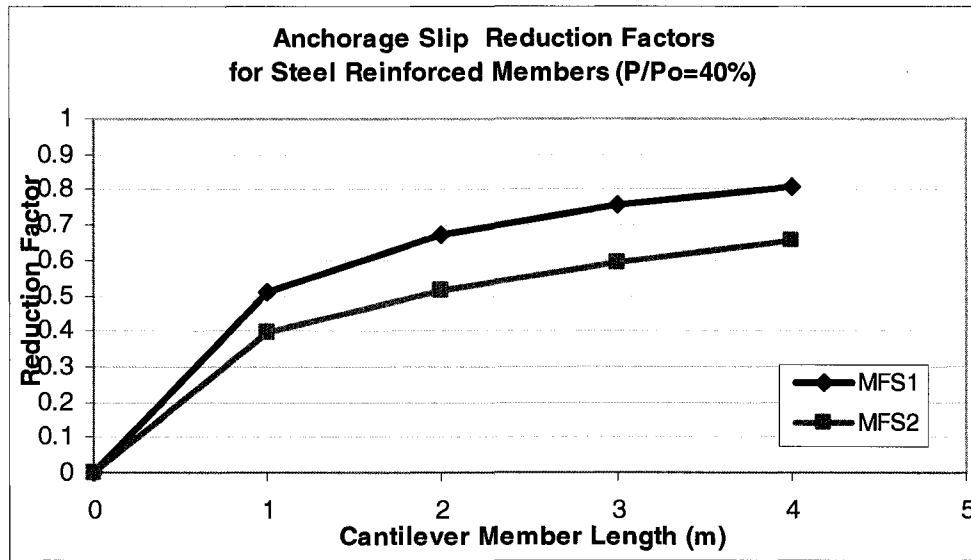


Figure 2.86 Slope reduction factors for steel reinforced sections with 40% axial loading

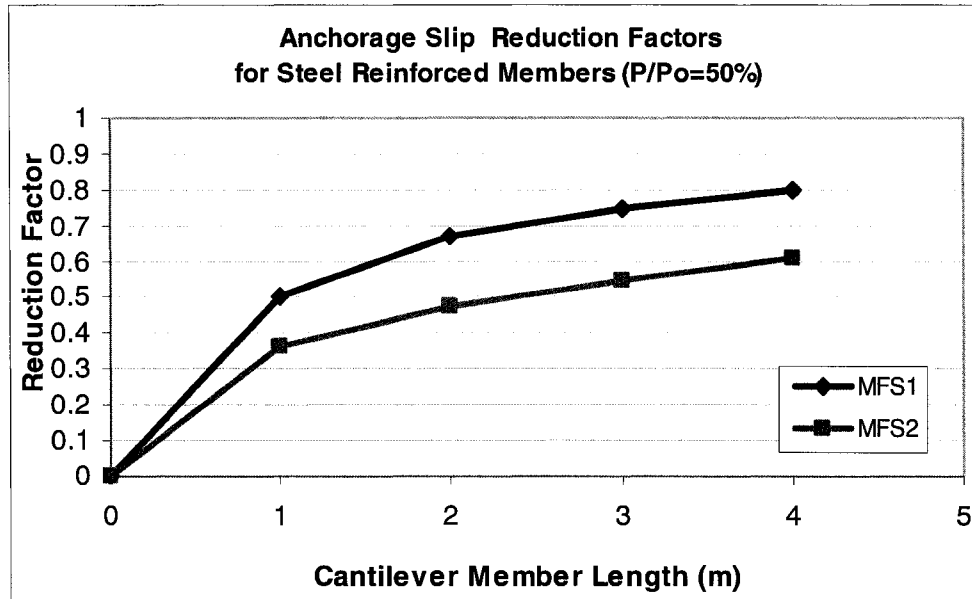


Figure 2.87 Slope reduction factors for steel reinforced sections with 50% axial loading

Chapter 3

Selection of Structures and Earthquake Records

3.1 Introduction

An important task in the current research program is to investigate seismic response of FRP reinforced concrete structures and to develop design recommendations based on the knowledge acquired through analysis. Therefore, a number of reinforced concrete buildings were selected for analysis. They were designed using FRP and steel reinforcement and analyzed using the computer software developed as part of the current investigation. This chapter describes the buildings selected and designed for analysis and ground motion records selected for dynamic analysis.

3.2 Selection of structures

Two sets of buildings were selected and designed with FRP reinforcement for Ottawa and Vancouver, representing regions of medium and high seismicity in Canada. Each set of buildings is consisted of 5, 10 and 15 storey-frame buildings as representatives of low-rise, medium-rise and high-rise buildings, recognizing that height plays an important role on dynamic characteristics (periods of vibration) and response of buildings. This resulted in three buildings for a given location. Additionally, six more companion steel reinforced concrete buildings were designed in order to conduct a comparative study between FRP and steel reinforced buildings.

A symmetric rectangular floor plan was selected for each building in order to eliminate the effects of torsion. Buildings designed for Vancouver have an additional frame in the short direction relative to the buildings designed for Ottawa. The additional frame (and hence shorter span lengths) had to be introduced to reduce lateral drift without using shear walls to keep deflections within the drift limit specified in the National Building

Code of Canada (NBCC-2005). The plan views of the 5, 10 and 15 storey moment resisting frame buildings in Ottawa and Vancouver are illustrated in Fig. 3.1. Fig. 3.2 illustrates the elevation view of buildings in the short direction for Vancouver location, which are also representative of building in Ottawa, except for the span lengths.

3.2.1 Modeling of structures for structural analysis

The framing system for buildings were modeled as series of two dimensional plane frames for the purpose of structural analysis, both for static analysis used in design and for dynamic analysis used to investigate seismic performance. This was decided even though the computer program developed and employed has three-dimensional analysis capabilities, because the hysteretic models adopted in the program do not consider bi-directional effects.

The structures selected for Ottawa and Vancouver had three and four interior frames in the short direction, respectively, and they both had two exterior frames. The properties of identical frames were lumped in one frame to reduce the number of elements for analysis. Consequently, there were two types of frames, one having the lumped properties of exterior two frames and the other having the lumped properties of three or four interior frames. These two lumped frames were then connected to each other with rigid beam elements to impose equal lateral displacement to simulate rigid floor diaphragms. The rigid link beams had infinitely high axial rigidities to ensure equal lateral displacement in the linked frames and infinitely small flexural rigidities to prevent moment transfer between the frames. A two dimensional lumped frame model of a 5-storey building selected for analysis is shown in Fig. 3.3 as a sample model. Structural mass was assumed to be concentrated at each floor level, and assigned to the nodes.

3.2.2 Effective stiffness and slopes of primary curve

The effects of potential cracking in members are taken into account by reducing their flexural rigidities. The flexural rigidities of steel reinforced columns and beams were reduced to $0.7 EI_g$ and $0.35 EI_g$ respectively based on CSA A23.3 (2004). Fig. 3.4, which is adapted from MacGregor and Bartlett (2000) shows the uncracked, cracked and

effective rigidities of a concrete section. The slope of force-deformation relationship that defines the effective rigidity of a section generally intersects with the moment-curvature curve at service load level where the moment is approximately 50% of the ultimate moment. Strain hardening ratio of steel reinforced members can be selected as 5 % of effective elastic rigidity. The ratio of the two end moments ζ , is taken 1.0 as described in Section 2.2.3.5. Effective elastic rigidity EI_{eff} , yield moment M_y which is obtained from the moment-curvature analysis, strain hardening ratio r and ratio of two end moments ζ are given as input data to the computer software SEQUAKE.

The effective stiffnesses of FRP reinforced concrete sections were also established analytically. A 300mm x 300mm square column, reinforced with 12 FRP bars with a bar diameter of 10mm was considered with 30MPa concrete. Two different levels of axial compression were used as 20% and 30% of column concentric capacity. The initial slopes of force-deformation relationships were plotted such that they passed through moment values corresponding to extreme concrete compressive strain of 0.001, as described in the model proposed by Saatcioglu and Sharbatdar (2003). These moments, expressed as M_1 in Figs 3.5 and 3.6 correspond to 50% and 55% of ultimate (where the concrete reaches its crushing strain) for the sections considered. The effective rigidities were found to be $0.4EI_g$ and $0.6EI_g$ under 20% and 30% of concentric compressive capacities, respectively. The concrete compressive strength was changed from 30MPa to 40MPa in the next analysis, with axial compression remaining at 30% of concentric capacity. In this case, the effective rigidity was found to be $0.55EI_g$ as depicted in Fig. 3.7. Based on the sectional analyses presented, it was concluded that the effective rigidity of FRP reinforced concrete columns can be taken as $0.50EI_g$.

Moment-displacement relationships were obtained through double integration of curvatures between the ends of members. For FRP reinforced concrete columns with 30 MPa concrete, it was found that the ratios of second segments to first segments defining the primary curve were 0.82 and 0.49 for columns subjected to 20% and 30% of concentric capacity, respectively. The same ratio was found to be 0.57 for a 40 MPa concrete column, subjected to 30% of its concentric capacity. The ratios of third

segments to first segments for the same columns were 0.56, 0.27 and 0.42. After examining the results of all the analyses conducted, it was decided that the ratios of second and third segments to first segments that define primary curves for FRP reinforced concrete columns can be taken as 0.7 and 0.5, respectively.

A similar investigation was conducted for FRP reinforced concrete beams. A T-beam with an effective width of 1500mm, web width of 300mm, total height of 400 mm and slab thickness of 150mm was selected with 2#25 top bars and 5#25 bottom bars. The concrete compressive strengths considered were 30MPa and 40MPa. The effective rigidities were found to be $0.14EI_g$ and $0.33EI_g$ for 30MPa and 40MPa concrete, respectively as illustrated in Figs.3.8 and 3.9. It was concluded that for FRP reinforced concrete beams, the effective rigidity can be taken as $0.2EI_g$ for the purpose of modeling primary force-deformation relationship. It was observed that the ratios of second and third segments to first segments on the primary curve of FRP reinforced beams were very large, implying almost perfectly elastic behaviour beyond the first segment, without much degradation due to concrete nonlinearity because of the reduced role that concrete plays in beams without axial compression. Therefore it was concluded that they can be taken as 1.0. However, in order to prevent numerical errors in the computer program employed, these ratios were taken as 0.99.

These slopes of primary force-deformation relationships were further reduced to allow for softening associated with anchorage slip, as discussed in Chapter 2. The ratio of two end moments ξ , was taken as 1.0 for both beams and columns with the implication that the points inflection are assumed to occur in the middle, with members bending in double curvature, for the purpose of computing inelastic deformations in hinges.

3.2.3 Seismic design forces

The design of the steel reinforced structure was carried out following the requirements of the CSA Standard A23.3 (2004), while the FRP reinforced concrete building was designed using the CSA Standard S806 (2002) and ISIS (2001) Manual. The buildings were analyzed with gravity and static seismic loads for design. The design base shears

were calculated according to the equivalent static load procedure of NBCC 2005, the use of which is permissible for the following classes of structures:

- Structures located in low-seismicity zones ($I_E F_a S_a(0.2) < 0.35$)
- Regular structures for which $h < 60\text{m}$ and $T_a < 2\text{ s}$
- Irregular structures for which $h < 20\text{ m}$ and $T_a < 0.5\text{ s}$ and are not torsionally sensitive.

Where; I_E is the importance factor, F_a is the acceleration based site coefficient, $S_a(0.2)$ is 5% damped spectral acceleration expressed as a ratio to gravitational acceleration for a period of 0.2 s, h and T_a are the height and the fundamental period of the structure. The following expression is used to determine the equivalent base shear of both steel and FRP reinforced structures:

$$V = \frac{S(T_a) M_v I_E W}{R_o R_d} \quad (3.1)$$

Where; $S(T_a)$ is the design spectral acceleration for fundamental period. The soil type was selected as very dense soil and soft rock (Site Class C). The factor M_v reflects higher mode effects on base shear and W is the total weight of structure. For the buildings considered, M_v was 1.0. The importance factor I_E was also taken as 1.0, since the buildings were considered as ordinary office buildings. The elastic base shear for steel reinforced concrete buildings is reduced by the product of the ductility and overstrength reduction factors, R_d and R_o , respectively. The R_d factor was taken as 2.5 for Ottawa and 4.0 for Vancouver, corresponding to nominally and fully ductile structures; while the overstrength factors were selected as 1.4 and 1.7 for buildings in Ottawa and Vancouver, respectively. The FRP reinforced buildings were designed for elastic seismic forces with $R_d = R_o = 1.0$. This was found necessary because of lack of inelasticity in FRP reinforcement and lack of dependable inelasticity in the structure, triggered by concrete confinement.

The fundamental period T was computed based on the empirical expression specified in NBCC-2005 for concrete structures:

$$T = 0.075(h_n)^{3/4} \quad (3.2)$$

More accurate estimation of the fundamental periods of structures was obtained by Eigen value analysis, using computer software SEQUAKE. In these analyses, the effects of potential cracking in members are taken into account for both steel and FRP reinforced concrete sections by reducing their flexural rigidities as described in Section 3.2.1. According to the NBCC (2005), the fundamental periods of the moment-resisting frames can not be taken greater than 1.5 times the value computed by Eq. (3.2); therefore adjustments were made when necessary in fundamental periods used for the calculation of equivalent base shear. The fundamental periods obtained from the NBCC (2005) expression and SEQUAKE are summarized in Table. 3.1

3.2.4 Structural design

The structures selected for the current investigation were designed following the NBCC (2005). Design load combinations were obtained from the same building code. Dead and live loads were selected as 5kN/m² and 2.4kN/m² for floors and 3.5kN/m² and 2.2kN/m² for roofs. Normal strength concrete, with $f'_c = 40\text{MPa}$ was used throughout the design. The longitudinal steel reinforcement was Grade of 400 MPa, while the FRP reinforcement was sand coated Carbon-FRP bars with a tensile rupturing strength of 1596 MPa. The columns and beams were confined with steel ties and hoops in the case of steel reinforced buildings and FRP grids in the case of FRP reinforced building to ensure confinement and inelastic deformability of concrete. CSA Standards A23.3-2004 and S806-2002 were employed for steel and FRP reinforced concrete buildings, respectively. In addition, the ISIS Design Manual (2001) was used for deflection requirements of FRP reinforced beams. Static analyses required for design were carried out using the computer program SEQUAKE. The analyses provided design values for axial forces, shear forces, bending moments and interstorey drifts.

The beam depth for steel and FRP reinforced concrete buildings were selected to satisfy the minimum thickness requirement defined in CSA Standard A23.3-2004 (2004) and

ISIS (2001), respectively. The strength requirements for steel reinforced beams were met by establishing the required reinforcement ratios to attain under-reinforced sections, as per CSA A23.3-2004. This resulted in top (negative) reinforcement that was larger than that required as bottom (positive) reinforcement. However, the same approach when applied to FRP reinforced beams led to the rupturing of bottom reinforcement prior to the attainment of concrete crushing in the positive moment region. The design approach adopted for FRP reinforced concrete buildings was to design them as over-reinforced sections to prevent failures by bar rupturing. This was felt essential because the buildings would be designed to sustain significant earthquake forces with potential inelasticity developing only in confined concrete to dissipate seismic induced energy. Therefore, the area of reinforcement was increased to impose concrete crushing in compression at failure. This created challenges as T-beams with large concrete compression zones needed very large amounts of FRP rebars to prevent bar rupturing. Top reinforcement was also checked and increased when necessary.

Column designs were performed based on the static analyses results. In this case, the strength requirements were generally sufficient for both steel and FRP reinforced concrete columns. In the Vancouver buildings, the interstorey drift limit governed the design of FRP reinforced columns. According to the NBCC (2005), the interstorey drift is limited to $0.025h_s$ for all buildings, other than post-disaster and school buildings for which this limit is reduced to $0.02 h_s$, where h_s is the storey height under consideration. Column sizes were increased to meet the drift limit, when necessary. The resulting member designs and design details are summarized in Figs. 3.10 to 3.13.

3.2.5 Damping

The damping assigned to each structure during dynamic analysis was depended only on stiffness. Kanaan and Powel (1973) state that the stiffness dependent damping would appear to be most reasonable for practical analysis of structures. Additionally, Wilson (1995) reported the over-damping effect of Rayleigh damping and noted: “The Newmark constant acceleration method, with addition of very small amount of stiffness proportional damping is recommended for dynamic analysis of nonlinear structural

systems.” Accordingly, the stiffness dependent damping was used for both steel and FRP reinforced structures, having 5% of critical damping.

3.3 Selection of earthquake records for seismic analysis

One of the primary objectives of the current investigation was to assess the seismic performance of FRP reinforced concrete frame buildings in Canada. Therefore, it is important to select ground motion records which are compatible with the Uniform Hazard Spectra (UHS) described in the NBCC (2005). NBCC (2005) defines the UHS by four site-specific spectral acceleration values at periods of 0.2, 0.5, 1.0 and 2.0 seconds and 5% damping. The UHS provides maximum design spectral acceleration values obtained by considering a range of earthquakes that contribute most strongly to the hazard at the specified probability level for the period values mentioned above. Consequently, the curve passing through these four parameters constructs the idealized UHS for each location with uniform hazard across the country. The probability of exceedance used in deriving UHS was 2% in 50 years.

Two different types of ground motion records were selected for seismic analysis. The first type was artificially generated by Atkinson and Beresnev (1998) for 10% in 50 year probability level. Subsequently, they developed a new set which matched 2% in 50 year probability level as specified in NBCC (2005). The researchers generated eight horizontal components for every selected city such that four represented a moderate earthquake nearby and the other four represented a stronger earthquake farther away. This was found necessary in order to match the moderate and strong distant earthquakes to short and long period energy bands. Events of moment magnitudes of M6.0 and M7.0 were selected to represent the short and long period hazards, respectively for eastern Canada. In the current study, Ottawa was selected to represent eastern Canada. For Ottawa, the hypocentral distances (R) were 30 km and 70 km for events M6.0 and M7.0, respectively. Vancouver was selected to represent western Canada. The eight artificial ground motion records generated by Atkinson and Beresnev (1998) were based on moment magnitudes of M6.5 and M7.2 for short and long period hazard, with hypocentral distances of 30 and

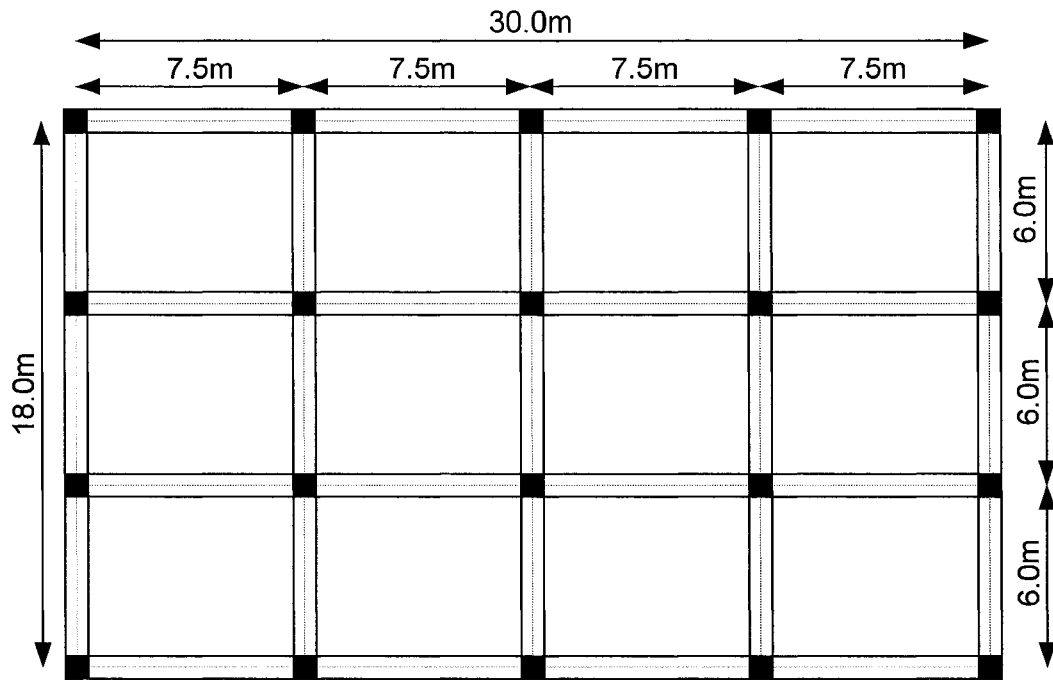
70 km, respectively. The resulting time histories were then scaled up or down by a 'fine tuning factor' in order to match the target spectra as much as possible.

In order to select UHS compatible earthquakes for Ottawa and Vancouver, the elastic response spectra were plotted for the selected actual earthquake records. These spectra were then compared with the design spectra defined by NBCC (2005). The ground motion with spectrum closest to the design spectrum for a given fundamental period of structure was selected for its time history analysis. It was observed that Short Event 4 in Ottawa and Short Event 4 in Vancouver governed all period ranges for the 5, 10 and 15 storey buildings.

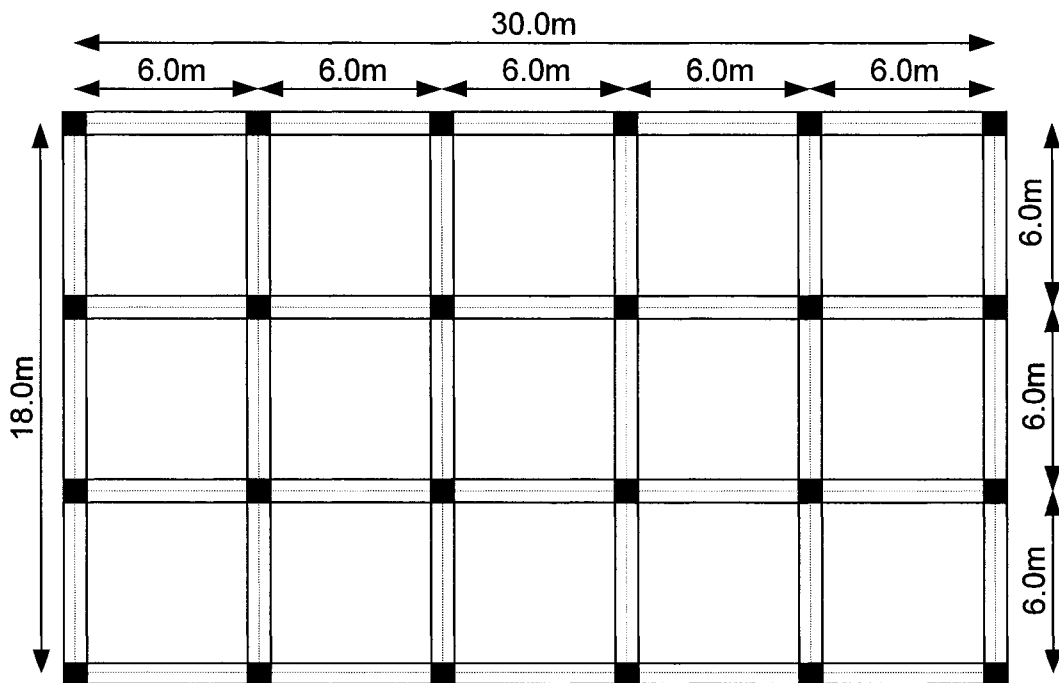
Additionally, Nisqually (Washington, 2001) and Tokachi Oki (Japan, 2003) earthquakes matching UHS for Vancouver city were selected for the analyses of buildings designed for Vancouver (Humar, 2005). These records were selected based on their spectral intensity and they were compared with that obtained from the UHS. After that, they were corrected or scaled using the recommendation proposed by Somerville et al. (1997) in order to minimize the error between the record and the UHS (Humar, 2005). The moment magnitudes of the earthquakes were 6.8 and 8.3 with hypocentral distances of 52 and 27 km, respectively. Nisqually (2001) record was used for the analysis of 5 storey building in Vancouver and Tokachi Oki (2003) record was used for the analysis of 10 and 15 storey buildings in Vancouver. Figs. 3.14 and 3.15 show the comparisons of response spectra for these earthquake records with the Uniform Hazard Spectra for Ottawa and Vancouver.

Table 3.1 Fundamental periods of buildings calculated using the NBCC-2005 expression and SEQUAKE

		STEEL REINFORCED BUILDINGS		FRP REINFORCED BUILDINGS	
Location	Number of Storey	T_{NBCC-2005} (sec)	T_{SEQUAKE} (sec)	T_{NBCC-2005} (sec)	T_{SEQUAKE} (sec)
Ottawa	5	0.71	2.03	0.71	1.9
	10	1.19	3.86	1.19	4.02
	15	1.62	5.1	1.62	5.26
Vancouver	5	0.71	1.5	0.71	1.47
	10	1.19	2.64	1.19	2.25
	15	1.62	3.65	1.62	3.48

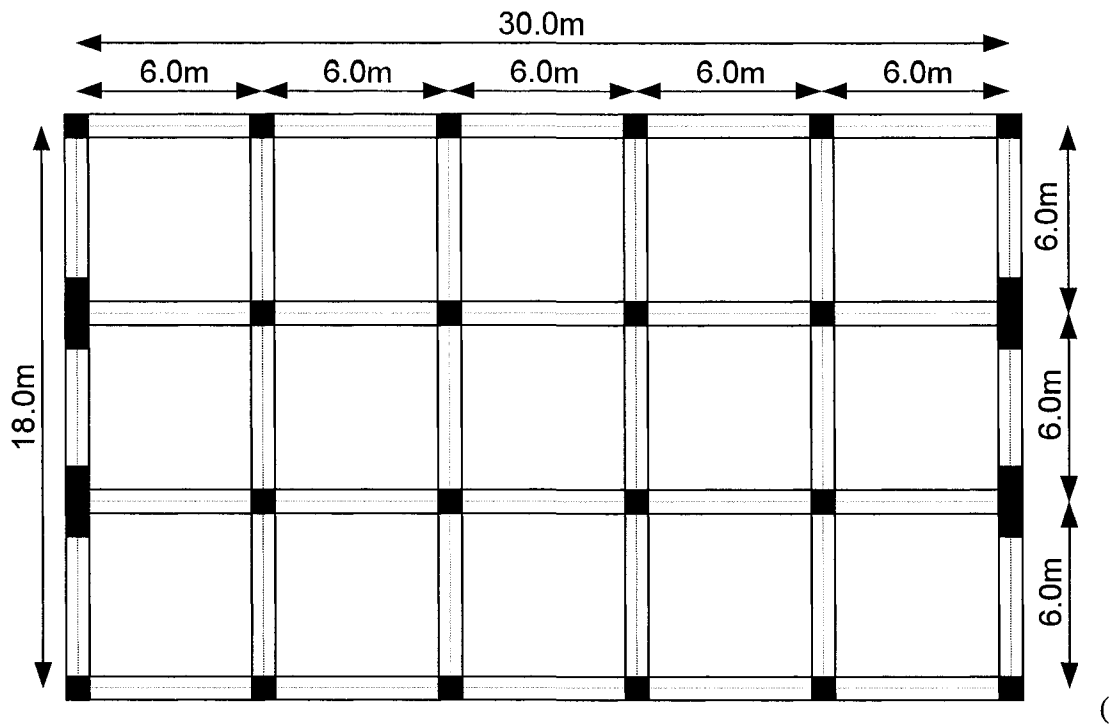


(a) Ottawa 5, 10 and 15 storey steel and FRP reinforced buildings

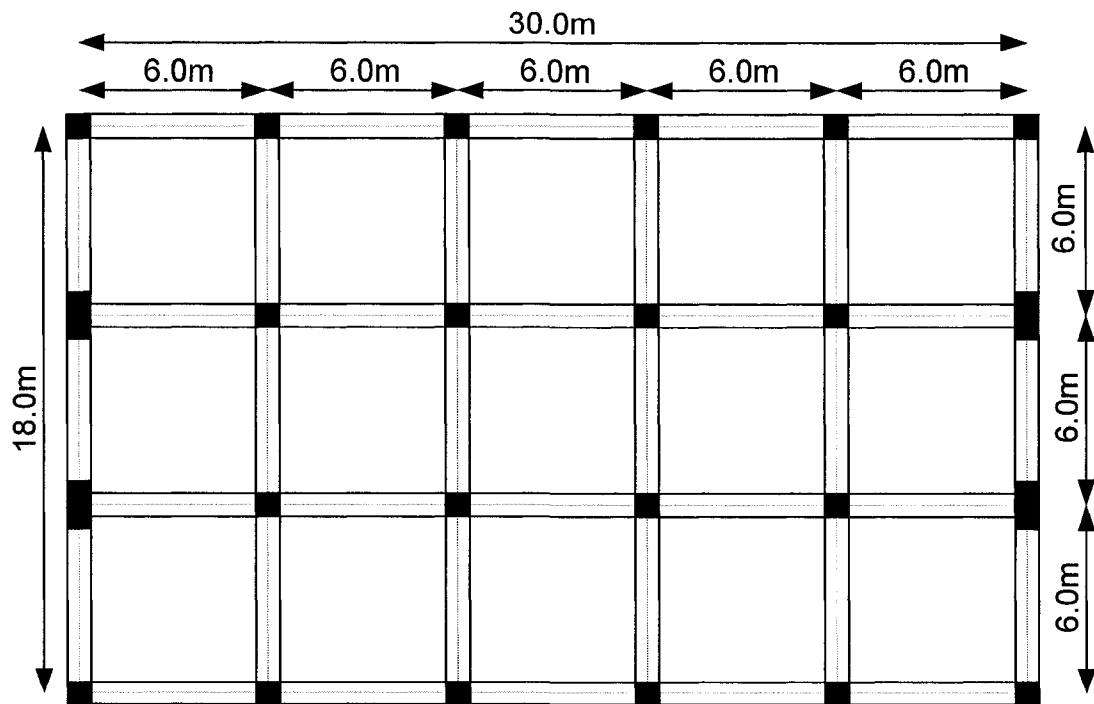


(b) Vancouver 5 and 10 storey steel reinforced buildings

Figure 3.1 Plan views of moment resisting frame buildings

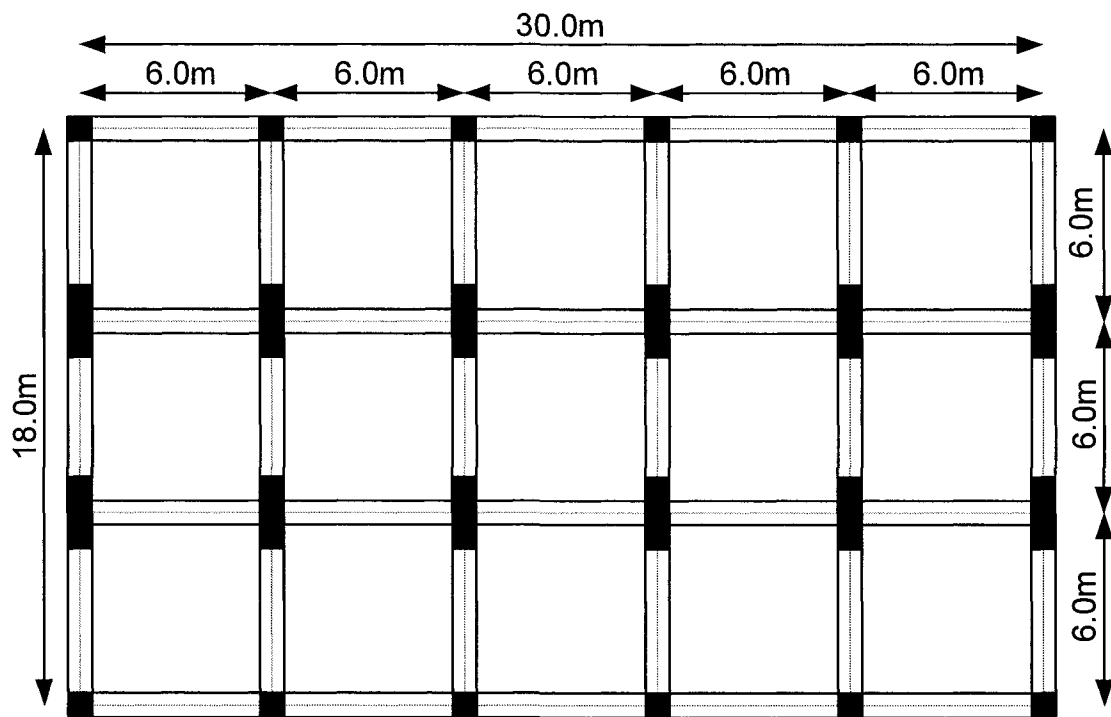


c) Vancouver 15 storey steel reinforced building



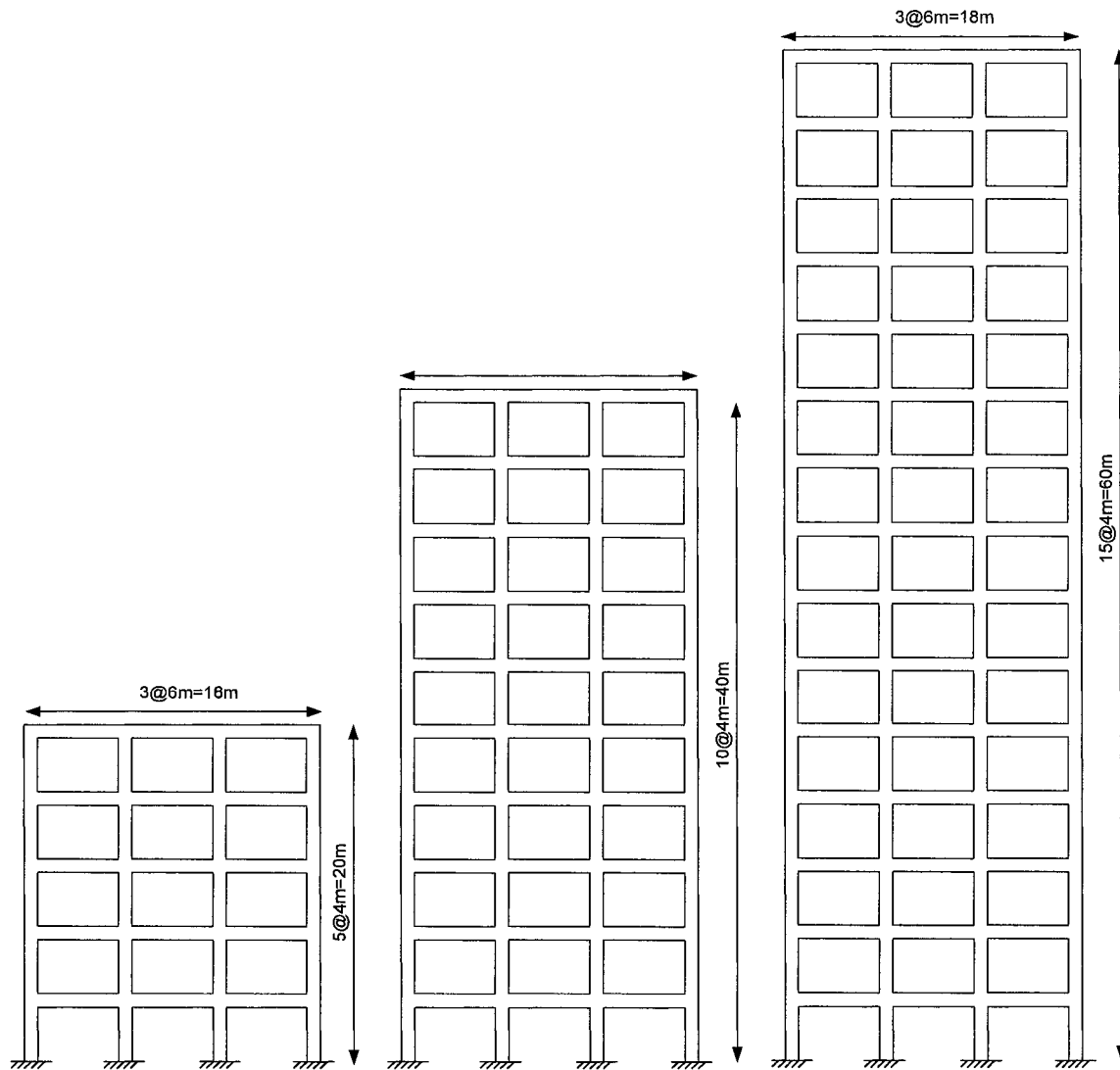
(d) Vancouver 5 storey FRP reinforced building

Figure 3.1 (Cont'd) Plan views of moment resisting frame buildings



(e) Vancouver 10 and 15 storey FRP reinforced buildings

Figure 3.1(Cont'd) Plan views of moment resisting frame buildings



(a) Ottawa 5, 10 and 15 storey steel and FRP reinforced buildings and Vancouver 5 and 10 storey steel reinforced building

Figure 3.2 Elevation views of frame buildings in the short direction



(b) Vancouver 15 storey steel and Vancouver 5, 10 and 15 storey FRP reinforced buildings

Figure 3.2 (Cont'd) Elevation views of frame buildings in the short direction

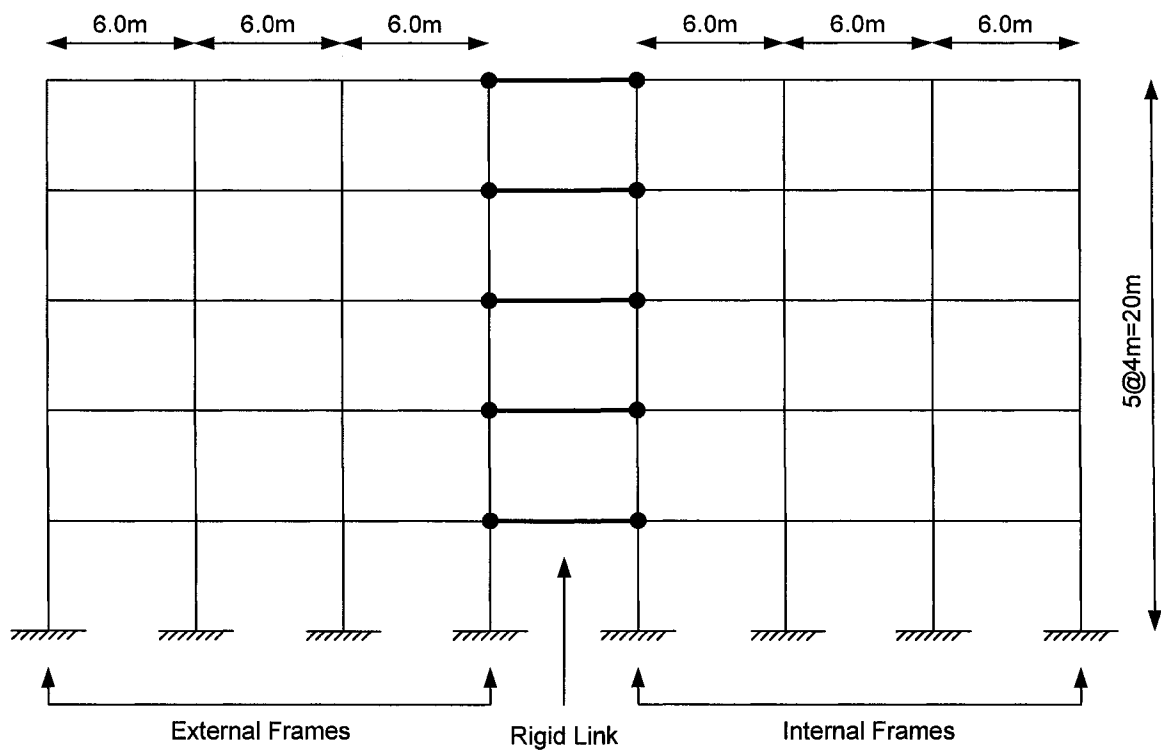


Figure 3.3 Two dimensional lumped 5 storey frame used in the analysis

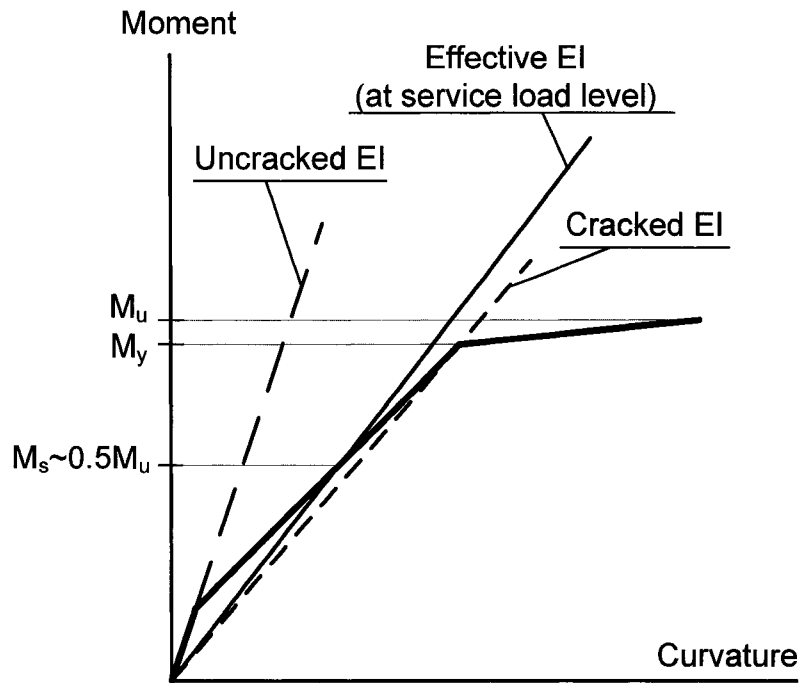


Figure 3.4 Effective stiffness of a reinforced concrete section
(Adapted from MacGregor and Bartlett, 2000)

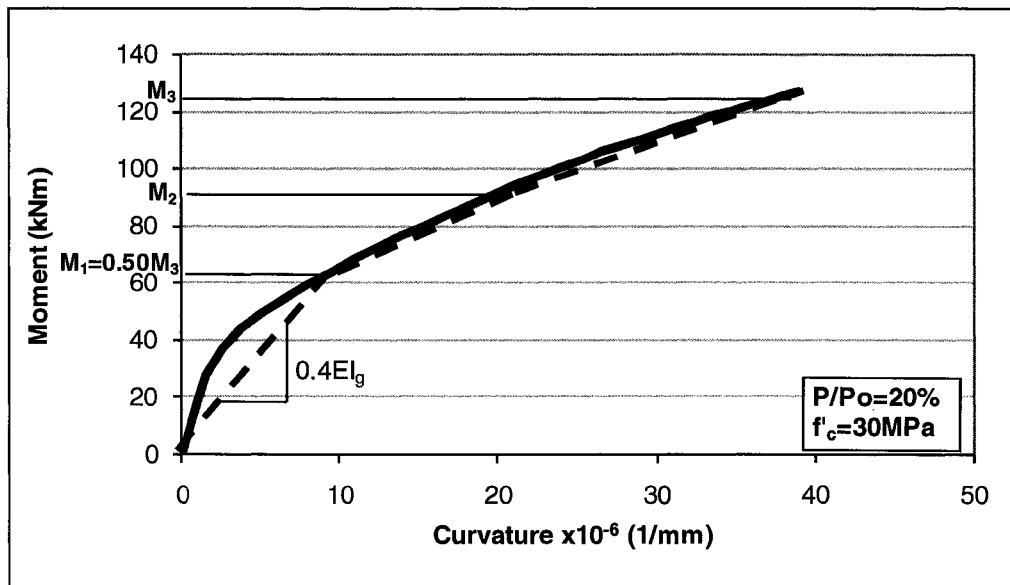


Figure 3.5 Moment-curvature relationship for a column with $P/P_o = 20\%$ and $f'_c = 30\text{MPa}$

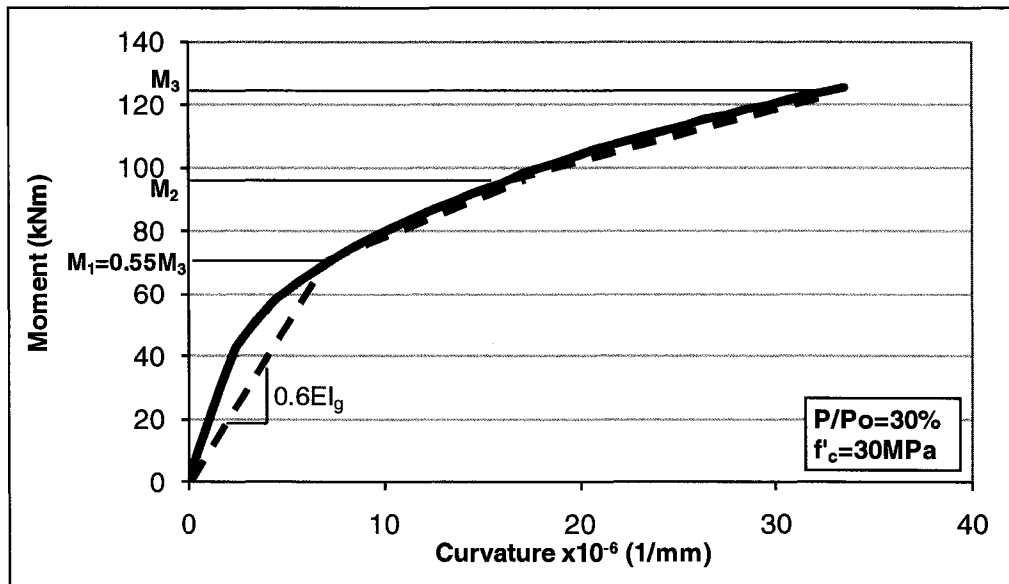


Figure 3.6 Moment-curvature relationship for a column with $P/P_o = 30\%$ and $f'_c = 30\text{MPa}$

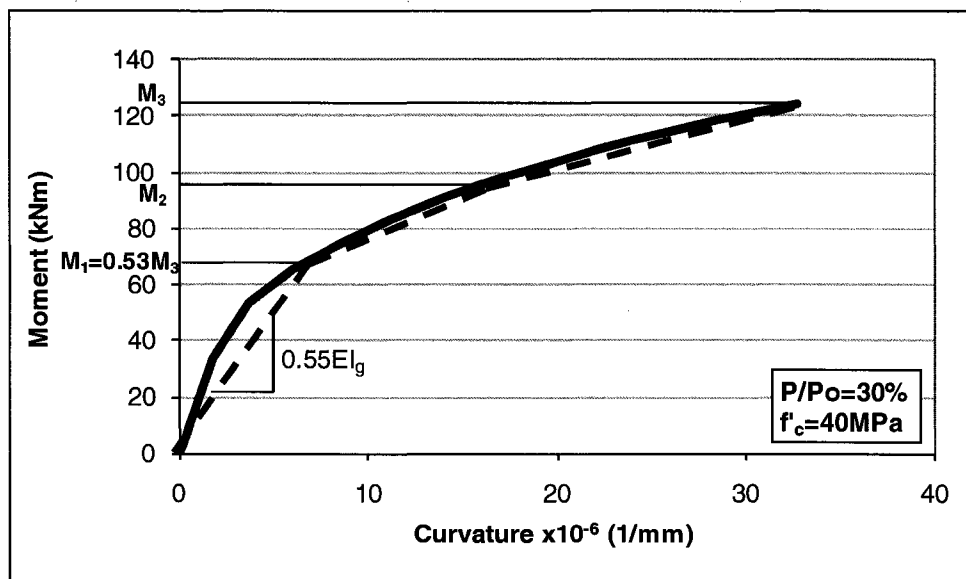


Figure 3.7 Moment-curvature relationship of the column with $P/P_o = 30\%$ and $f'_c = 40\text{MPa}$

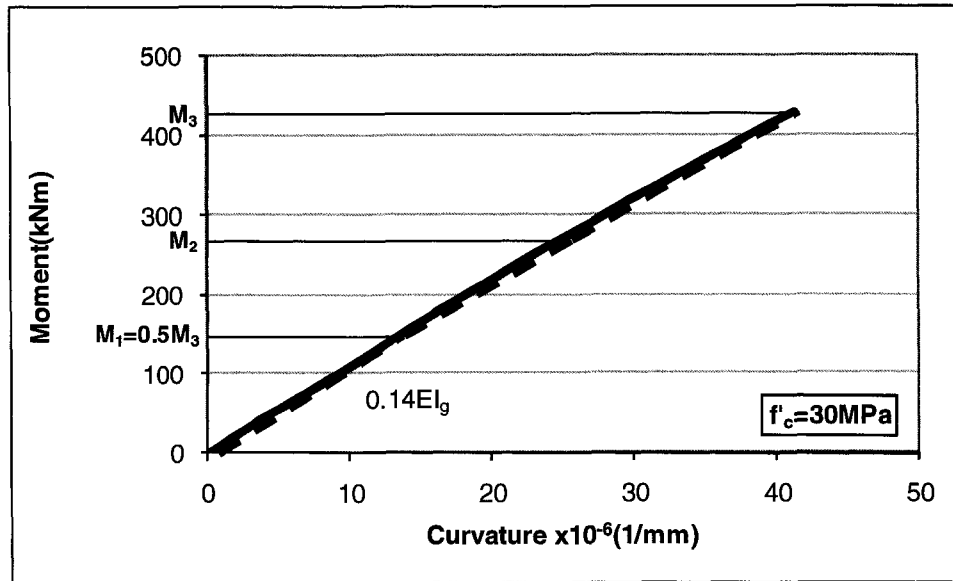


Figure 3.8 Moment-curvature relationship for a beam with $f'_c=30\text{MPa}$

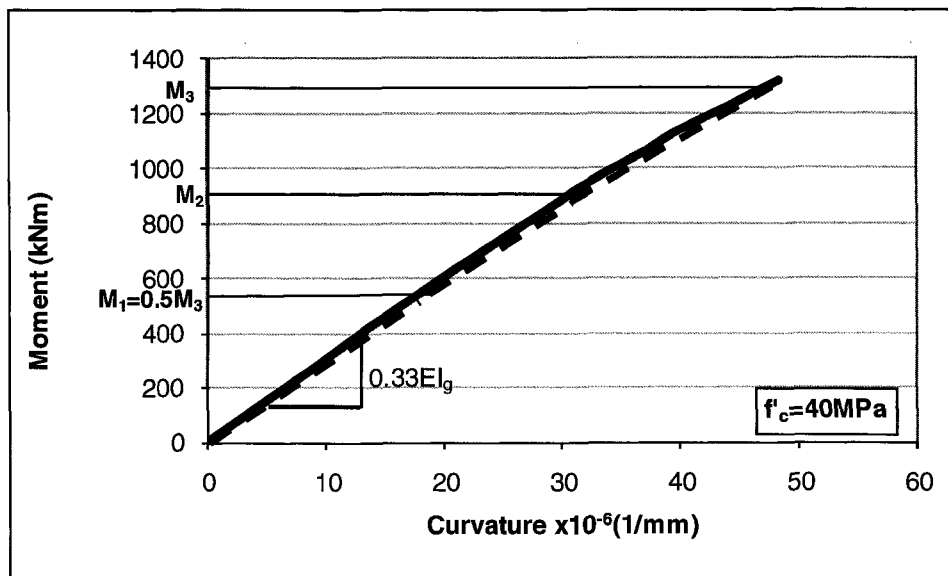


Figure 3.9 Moment-curvature relationship for a beam with $f'_c=40\text{MPa}$

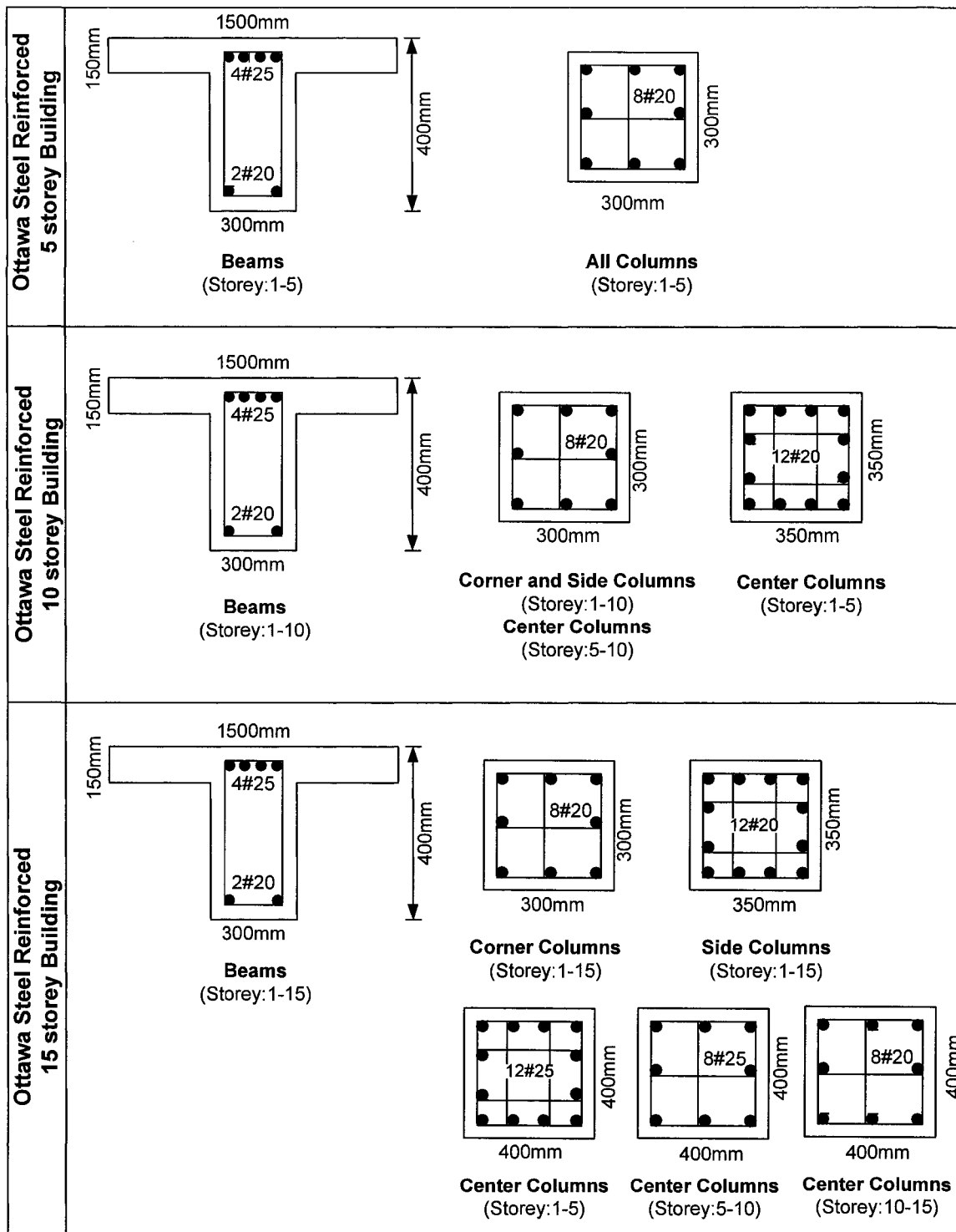


Figure 3.10 Sectional details of steel reinforced structures designed for Ottawa

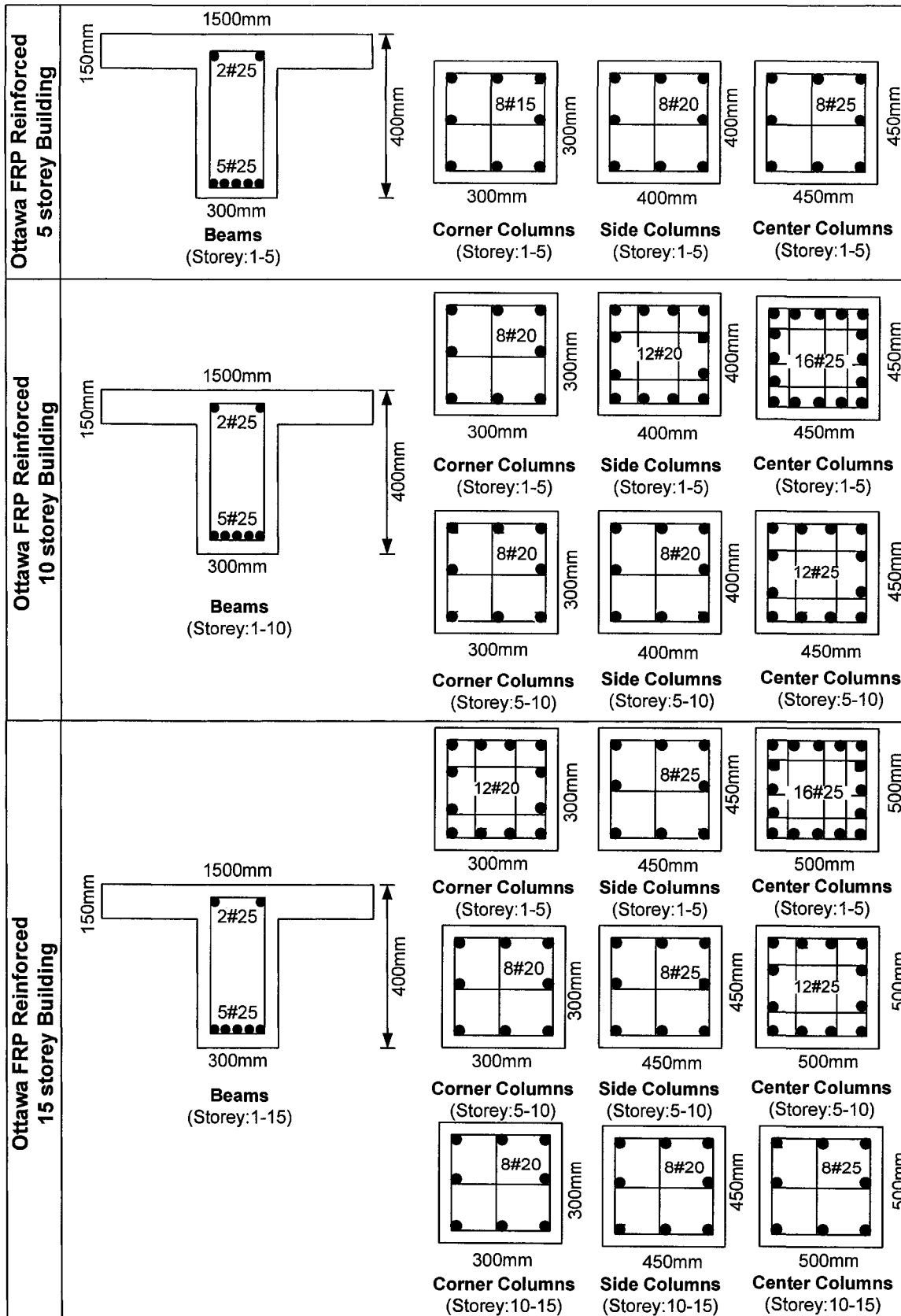


Figure 3.11 Sectional details of FRP reinforced structures designed for Ottawa

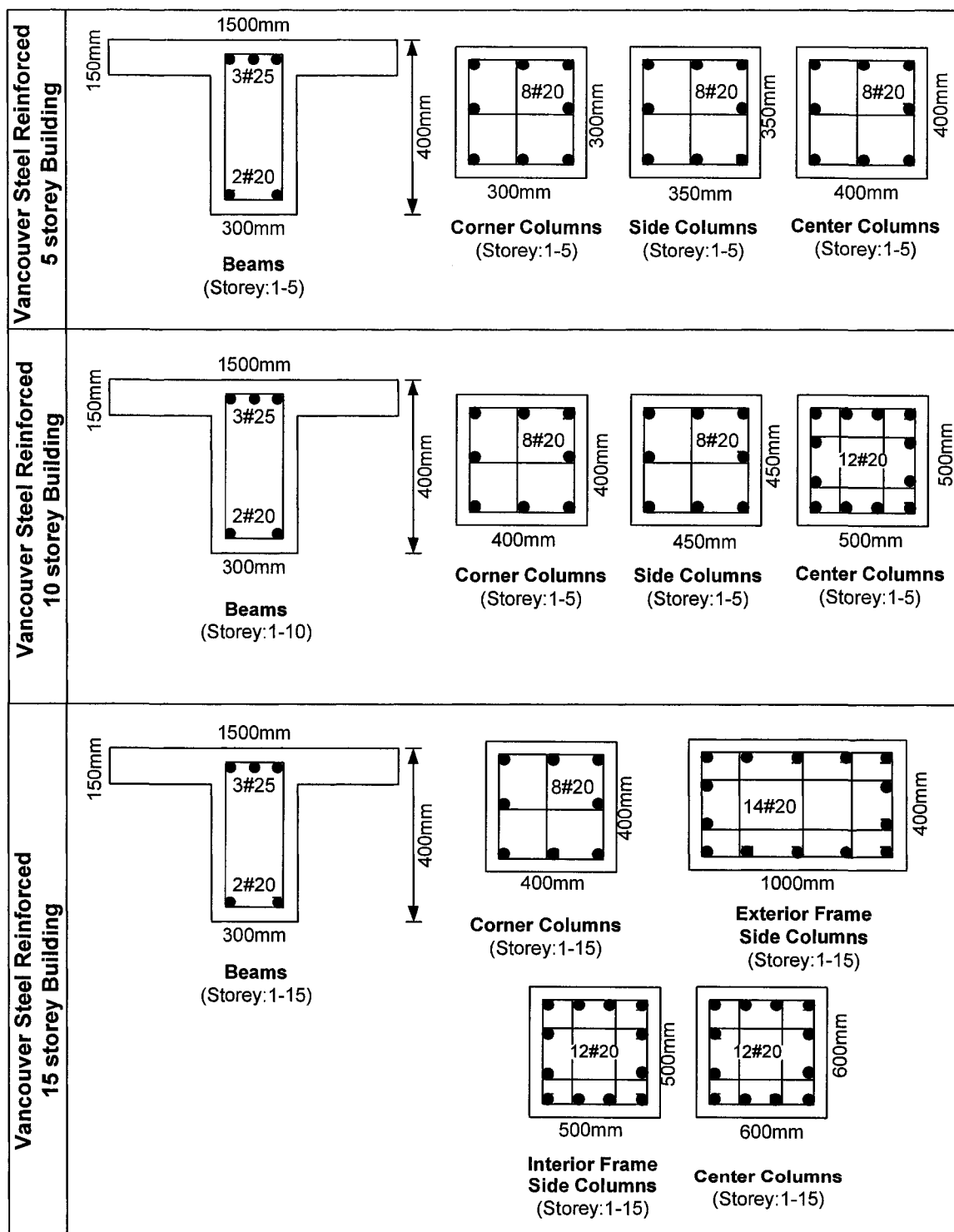


Figure 3.12 Sectional details of steel reinforced structures designed for Vancouver

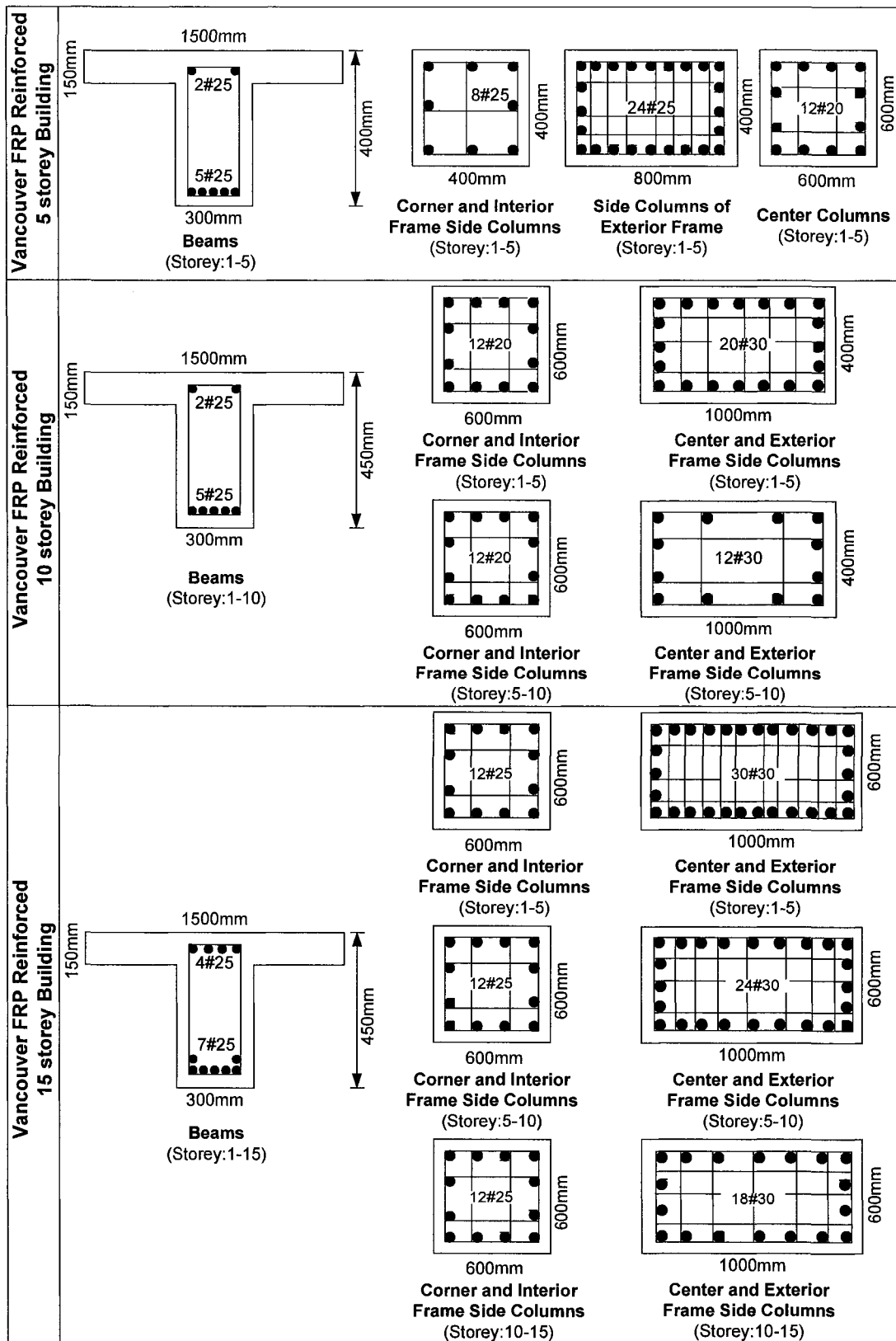


Figure 3.13 Sectional details of FRP reinforced structures designed for Vancouver

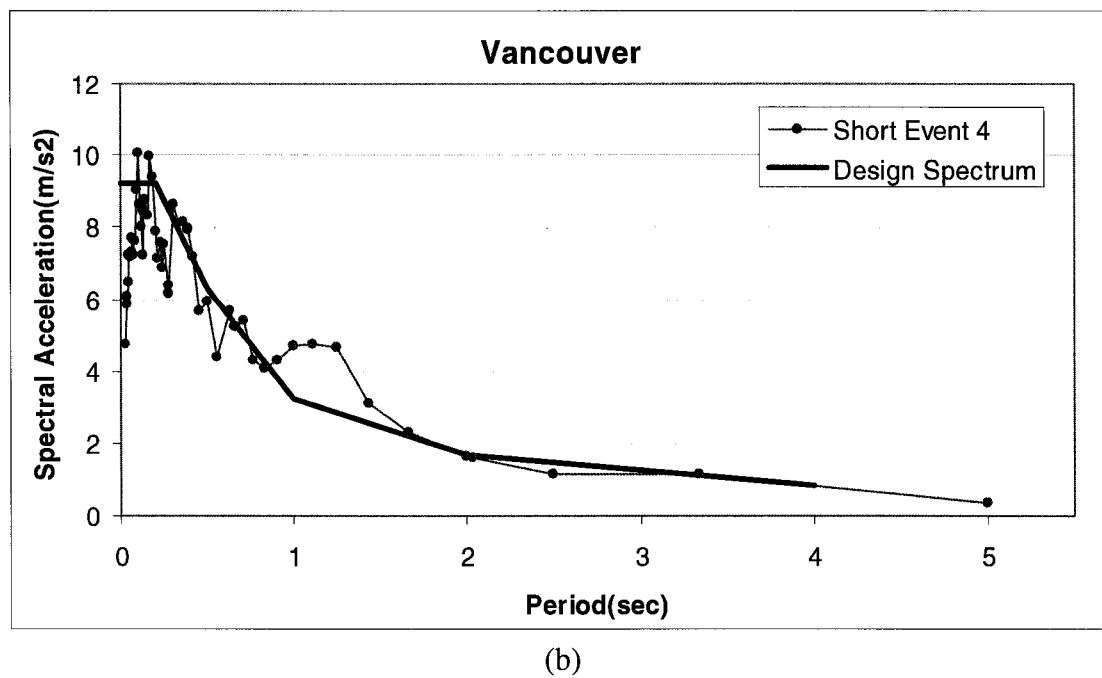
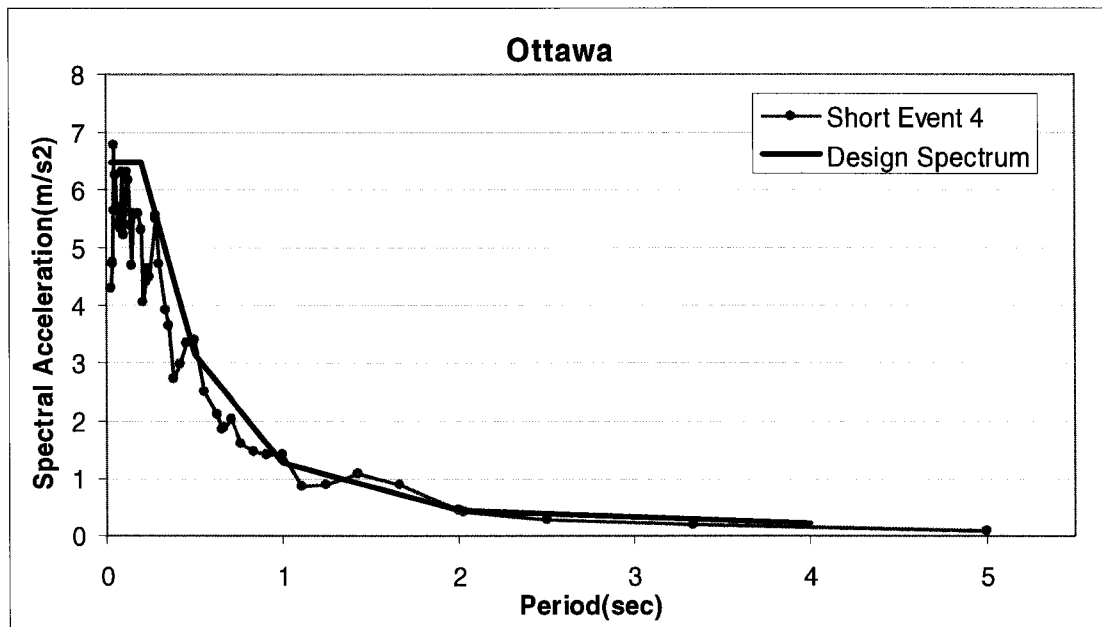
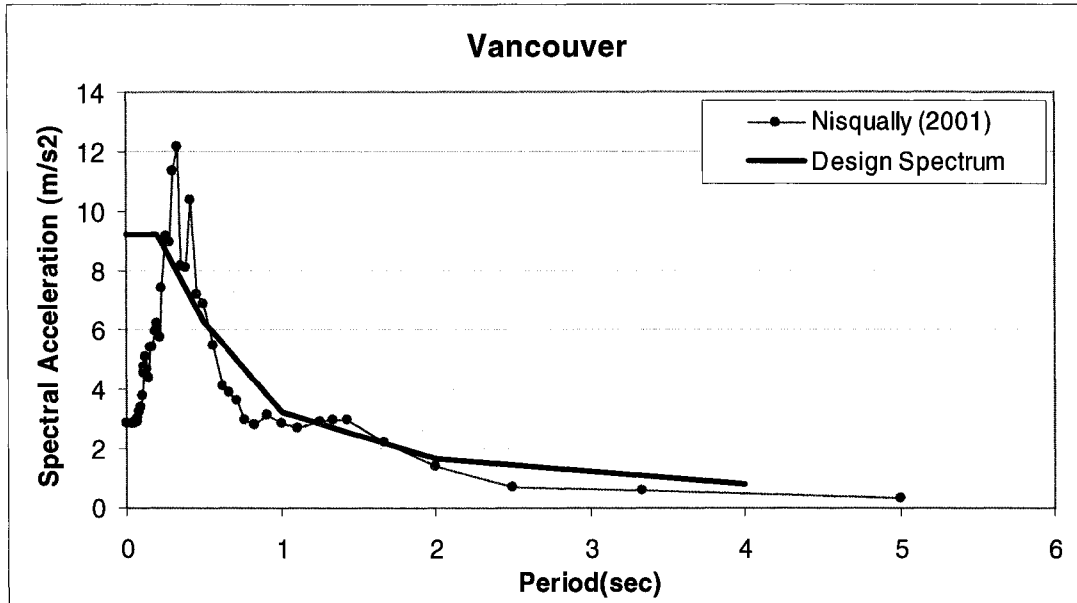
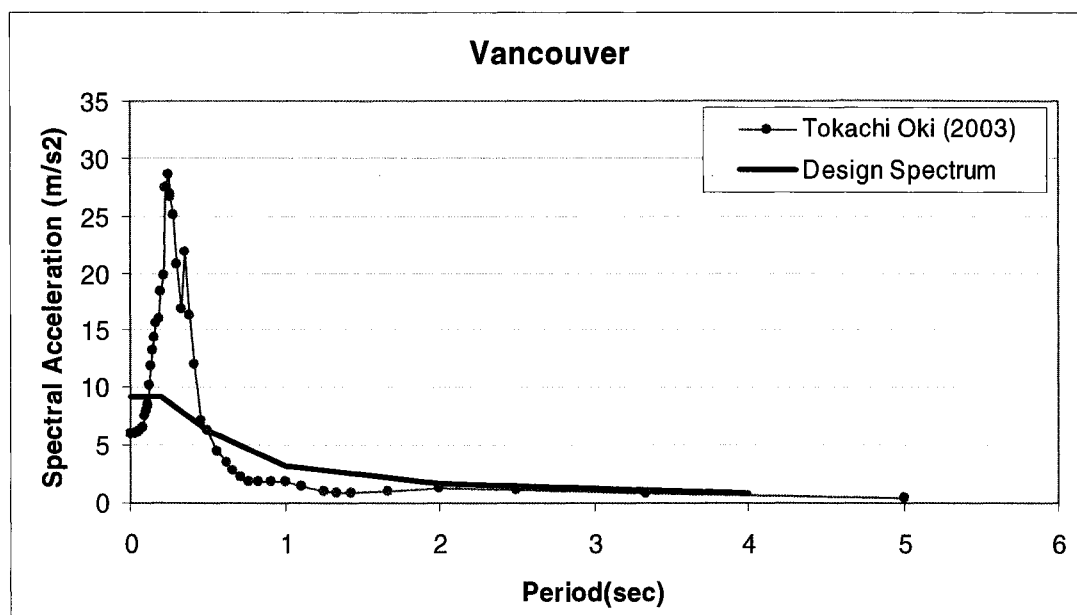


Figure 3.14 Comparisons of response spectra for artificially generated earthquakes and UHS (design spectrum) for Ottawa and Vancouver



(a)



(b)

Figure 3.15 Comparisons of response spectra for previously recorded earthquakes and UHS (design spectrum) for Vancouver

Chapter 4

Analysis Results

4.1 Introduction

One of the primary objectives of the current study was to investigate the seismic response of FRP reinforced concrete buildings, and comparison with companion steel reinforced concrete response. This was achieved by conducting dynamic inelastic response history analysis of the 5, 10 and 15 storey buildings selected and designed in Chapter 3. Two different types of earthquake records were used: previously recorded actual records and artificially generated records, both scaled to match the Uniform Hazard Spectra given in the NBCC (2005).

In addition, a specific set of analyses was conducted to assess the effects of anchorage slip on seismic response of FRP reinforced concrete buildings. The 5-storey buildings designed for Vancouver, with steel and FRP reinforcement, were analyzed for this purpose using previously recorded and artificially generated earthquake records. The same set of analyses was repeated under increased intensity of ground motion to assess the significance of inelasticity, induced by concrete confinement.

4.2 Seismic response of FRP reinforced concrete structures

The 5, 10 and 15 storey buildings designed for Ottawa and Vancouver in Chapter 3 were analyzed using the artificial earthquake records (Short Event 4 for Ottawa and Short Event 4 for Vancouver) generated by Atkinson and Beresnev (1998). The Vancouver buildings were also analyzed using previously recorded earthquake records; Nisqually (2001) earthquake was used for 5-storey building and Tokachi Oki (2003) was used for 10 and 15-storey buildings. The results are presented in terms of force and deformation

demands, and are compared with those obtained by analyzing companion steel reinforced concrete buildings.

4.2.1 Base shear

The results of dynamic inelastic response history analysis were evaluated in terms of base shears demands. Dynamic base shears were also compared with those obtained through equivalent static load analysis. All buildings analyzed in the current study had regular building layouts with periods less than 2 sec according to NBCC-2005 empirical formula and heights less than 60m. Therefore they met the NBCC-2005 requirements for equivalent static load procedure. The fundamental periods of both steel and FRP reinforced buildings obtained by dynamic analyses were much longer than those obtained with code-specified empirical expressions, as can be seen in Table 3.1. The code, however, does permit up to a 50% increase in empirical period values for the purpose of computing equivalent static forces. Therefore, the values obtained by the empirical expression were increased by 50% for the purpose of calculating static design base shears. One of the consequences of differences in computed periods is the possibility of obtaining reduced shear demands through dynamic analysis. The code does permit a reduction in static seismic base shear if the base shear obtained from dynamic analysis is lower, provided that the resulting base shear is not less than 80% of the value obtained by the equivalent static load procedure. This reduction was utilized in the current investigation, in obtaining static design base shear demands. The maximum base shear values obtained from the equivalent static load procedure and dynamic analyses are compared in Table 4.1. For buildings in Ottawa, the maximum base shears obtained from inelastic dynamic analyses of steel reinforced buildings were less than those obtained through equivalent static force procedure used in design, but they were close, and these buildings did not experience inelasticity. Maximum base shears obtained through dynamic analysis for FRP reinforced buildings were approximately the same as those for steel reinforced buildings, even though the static base shear values for FRP reinforced buildings were significantly higher than those for steel reinforced buildings because of the low $R_d R_o$ product used for the FRP buildings. This was expected because both steel and FRP reinforced structures remained elastic, well below the design force levels for steel reinforced buildings.

For buildings in Vancouver, while dynamic base shear demands for steel reinforced structures were higher than those static base shears considered in design, the opposite was observed for FRP reinforced structures. The maximum base shears obtained using previously recorded earthquakes were larger than those obtained using the artificially generated records. Steel reinforced buildings experienced nonlinearity, while the FRP reinforced buildings remained elastic without developing failure strengths as incorporated into the hysteretic model used. Base shear responses of buildings are shown in Figs. 4.1 to 4.18.

4.2.2 Drift ratios

Maximum interstorey drift ratios obtained from static and nonlinear dynamic analyses are summarized in Table 4.2. In Ottawa, the same pattern is observed for drift ratios as that for the base shears. The maximum interstorey drift ratios for steel reinforced buildings obtained from nonlinear analyses are close to those obtained from elastic analyses. They were also approximately the same as those obtained from dynamic analyses of FRP reinforced buildings. The dynamic drift ratios for both steel and FRP reinforced structures in Ottawa were significantly below than those computed under equivalent static design forces.

In Vancouver, since the steel reinforced buildings experienced nonlinear deformations, their maximum drift ratios were higher than those computed on the basis of *elastic* seismic forces. The drift ratios obtained using previously recorded earthquakes were larger than those obtained from artificially generated records for 5 and 10 storey buildings, though the opposite was true for the 15 storey building. All drift ratios were within the NBCC (2005) limit of 2.5 %. The interstorey drift ratios for FRP reinforced buildings obtained through dynamic analyses were less than those obtained from static analyses, except for the 5 storey building subjected to previously recorded earthquakes, in which case the drift values were almost the same. Drift ratios for all FRP reinforced buildings were below the NBCC (2005) limit of 2.5 %.

The maximum interstorey drifts of FRP reinforced buildings are compared with those for steel reinforced buildings designed for Ottawa and Vancouver in Figs. 4.19 to 4.27. The roof displacement time histories of buildings are shown in Figs. 4.28 to 4.45. The results indicate that, while responses vary between steel and FRP reinforced concrete buildings based on their dynamic properties and the interaction of structural and ground motion frequencies, there is no pattern observed in terms of one being consistently above or below the other. The interstorey drift ratios and absolute roof displacements were similar but different for FRP and steel reinforced buildings, all below the code maximum limit, showing that FRP reinforced concrete buildings can be designed by following the same design concepts currently utilized for steel reinforced concrete buildings, while recognizing the differences in material behaviour and associated R_d and R_o factors.

4.2.3 Moment-chord rotation relationships

Moment-chord rotation relationships for critical elements are examined to assess the level of inelasticity developed in each building, during dynamic response. All the steel reinforced buildings designed for Ottawa remained elastic. The beams of 5, 10 and 15 storey buildings developed a maximum of 77%, 70% and again 70% of their yield moments, respectively. The columns developed a maximum of 43%, 28% and 24% of their yield moments for 5, 10 and 15 storey buildings, respectively.

The FRP reinforced concrete buildings were modeled such that the beam flexural capacity would be reached through the crushing of concrete, with little inelasticity allowed in hysteretic relationships due to the confinement of concrete, which plays a small role on beam response. The beams of 5, 10 and 15 storey FRP reinforced buildings designed for Ottawa developed a maximum of 28%, 25% and 31% of their failure moments respectively. The columns developed a maximum of 18%, 12% and 10% of their failure moments in 5, 10 and 15 storey FRP reinforced buildings, respectively. The moment-rotation relationships for sample first storey beams and columns of steel and FRP reinforced buildings in Ottawa are plotted in Figs. 4.46 to 4.51, showing elastic response.

The steel reinforced 5-storey building designed for Vancouver experienced nonlinearity in both the beams and columns. The analyses using artificially generated and previously recorded earthquakes respectively gave maximum ductility ratios of 2.9 and 2.8 for the beams and 2.4 and 2.1 for the columns. In the 10 storey building designed for Vancouver, the maximum ductility ratios were 2.1 and 2.3 for the beams and 1.3 and 1.4 for the columns under artificially generated and previously recorded earthquakes, respectively. In the 15 storey Vancouver building, the maximum ductility ratios were 1.6 and 2.0 for the beams and 1.5 and 2.0 for the columns under artificial and recorded earthquake ground motions, respectively.

FRP buildings in Vancouver did not experience failure in any member. In the 5 storey building, the beams developed a maximum of 63 % and 72 % of their failure moments when artificially generated and previously recorded earthquakes were used, respectively. The rectangular columns used in exterior frames developed a maximum of 70 % and 96 % of their failure moments, respectively under the same two earthquakes. The maximum moments developed in the remaining square columns were 51% and 72% of their failure moments when artificially generated and previously recorded earthquakes were used, respectively. In the 10 storey FRP reinforced building the beams developed a maximum of 35 % and 38 % of their failure moments when artificially generated and previously recorded earthquakes were used, respectively. The columns developed a maximum of 29 % and 34 % of their failure moments under the same earthquake records. In the 15 storey FRP reinforced building the beams developed a maximum of 56 % and 65 % of their failure moments, and the columns developed a maximum of 35 % and 44 % of their failure moments when artificially generated and previously recorded earthquakes were used, respectively. Some of the first storey beam and column hysteretic moment-chord rotation relationships for steel and FRP reinforced buildings in Vancouver are illustrated in Figs. 4.52 to 4.63.

4.3 Effect of anchorage Slip

Anchorage slip, as characterized by the extension of anchored reinforcement within the adjoining member, may result in the softening of members and a reduction in structural

stiffness, potentially affecting dynamic characteristics and structural response to earthquakes. This effect is not included in flexural analysis, because the flexural analysis assumes rigidly connected elements, ignoring potential softening of elements at the joints. Anchorage slip in steel reinforced concrete buildings can be significant because of the penetration of yielding at the face of the member into the adjoining member, especially if the steel enters into the strain hardening region. FRP reinforcement does not develop inelasticity and this effect may be perceived to be small. However, the elastic moduli of FRP bars are often significantly lower than that of steel, potentially producing significant elastic elongation within the adjoining member giving rise to anchorage slip.

The softening effect of anchorage slip in buildings and their seismic response were investigated, and are discussed in this section. The 5-storey Vancouver buildings, with steel and FRP reinforcements, were selected for analysis. The amount of softening due to anchorage slip was calculated and modeled as explained in Chapter 2. Each building was analyzed with and without anchorage slip. The buildings were subjected to the same ground motions selected for Vancouver as before. Additional analysis was conducted using another artificially generated earthquake by Atkinson and Beresnev (1998): 'Long Event 1 for Vancouver,' to investigate the significance of the frequency content of earthquake records.

Maximum base shears obtained from the analyses are summarized in Table 4.3. They indicate that in the steel reinforced structure, when the anchorage slip effect was considered, the maximum base shear decreased approximately by 15% to 25% as compared to the building where this effect was neglected, except for the analysis under Long Event 1, which did not show any significant effect on base shear. A similar trend was observed for the FRP reinforced building. When 'Short Event 4 for Vancouver' and Nisqually Earthquake Record (2001) were used as ground motions, there were 13% and 23% reductions in maximum base shear due to the incorporation of anchorage slip in analysis. However, the opposite was true for the Long Event 1 Artificial Record which showed an increase of 13% in base shear due to anchorage slip.

The effect of anchorage slip on maximum interstorey drift is shown in Table 4.4 for both steel and FRP reinforced buildings. The maximum interstorey drift ratio for the steel reinforced structure increased by 12% and 35 % when artificially generated records were used and anchorage slip was considered. However, the drift ratios remained approximately the same when the Nisqually Record (2001) was used. In the FRP reinforced structure, the artificially generated earthquakes created 7% and 63% increase in maximum interstorey drifts when the anchorage slip effect was considered, however there was a 14% decrease when Nisqually (2001) earthquake was used. The maximum interstorey drift ratios were illustrated in Figs. 4.64 to 4.69 for both steel and FRP reinforced structures. The maximum effect of anchorage slip was observed in both steel and FRP reinforced structures when ‘Long Event 1’ was used as the ground motion. In all cases, the maximum interstorey drift ratios remained within the limit of 2.5% specified in the NBCC (2005).

Nonlinear deformations were observed in steel reinforced buildings, irrespective of the consideration of anchorage slip in analyses. However, the ductility demand decreased by a maximum of 36 % in beams and by a maximum of 48% in columns when anchorage slip was considered, except when Long Event 1 was used as earthquake record, in which case the drift demand was not affected. In FRP reinforced structures, when ‘Short Event 4’ and Nisqually (2001) records were used, the ratios of the moments developed in sections to their failure moments were decreased by a maximum of 13% in beams and by a maximum of 46% in columns. When ‘Long Event 1’ was used, the maximum moment attained relative to moment capacity did not change for the beams, though the column moments increased up to 33% when anchorage slip was considered. Sample moment-rotation hysteretic relationships are shown for selected beams and columns, with and without the consideration of anchorage slip in Figs. 4.70 to 4.75 for comparison. In general, it can be concluded that anchorage slip does affect dynamic response of structures considerably. The same conclusion also applies to FRP reinforced structures, even though FRP reinforcement behaves elastically. Hence it is recommended that potential softening in flexural response, caused by anchorage slip be included in seismic analysis.

4.4 Effects of nonlinearity in response

The earthquake records used in previous sections were scaled to have peak ground accelerations for a specific probability of occurrence, i.e. 2% in 50 years. However, different hazard levels with different intensities do occur randomly. For the investigation of seismic performance of FRP reinforced concrete buildings, it becomes especially important to study the performance beyond the assumed design hazard level to ensure life safety. FRP reinforcement is a new material for use in earthquake resistant construction, with no previous experience and brittle material characteristics. Therefore, the understanding of structural response beyond the elastic range of deformations is very important for the acceptance of this new material in the construction industry. This was the rationale that triggered further investigation of inelastic response, especially of FRP reinforced structures even though they are intended to remain elastic during seismic response. For this reason, the intensity of earthquake records used in Section 4.3 were increased by 100% and used for the analysis of 5 storey building designed for Vancouver to trigger or increase inelasticity. A number of analyses were conducted to compare linear and nonlinear response, with and without the effects of anchorage slip, for both steel and FRP reinforced structures.

The effects of nonlinear response on maximum base shear are summarized in Table 4.5. In steel reinforced structures, when anchorage slip was neglected, there was a decrease of 61% to 73% in maximum base shear due to the nonlinearity considered. The decrease was of 36% to 67% when the same comparison was conducted with anchorage slip. In FRP reinforced concrete structures, nonlinearity resulted in 18% and 50% reduction in based shear under intensified ‘Short Event 4’ and Nisqually (2001) earthquake records, without the consideration of anchorage slip. The same comparison indicated a 16% increase when intensified ‘Long Event 1’ was used as the earthquake record. The incorporation of anchorage slip in analysis resulted in 6% to 35% reduction in base shear due to nonlinearity under the three ground motions used.

The maximum interstorey drift ratios obtained from linear and nonlinear analyses, with and without anchorage slip, are summarized in Table 4.6. In steel reinforced structures,

when anchorage slip was neglected, there was a decrease of interstorey drift of up to 50% due to the consideration of non-linearity. In FRP reinforced concrete structures, when the anchorage slip effects were ignored, the maximum interstorey drift ratios either increased by 18% to 100% or they remained approximately the same with the consideration of inelasticity in analysis. When the analyses were repeated with the consideration of anchorage slip, the steel reinforced concrete building either showed approximately 33% to 45% decrease in drift with inelasticity or remained approximately the same. The FRP reinforced building developed up to 38% increase due to the inelasticity of deformations with the consideration of anchorage slip. The comparisons indicate that, while in general drift ratios increased with inelasticity, there was no consistent pattern observed. The structural response changed considerably by the consideration of inelasticity in response as a result of the change in dynamic characteristics of buildings and the interaction with the frequency content of exciting ground motion.

The maximum interstorey drift ratios are illustrated in Figs. 4.76 to 4.87. The maximum interstorey drift ratios for steel reinforced structures obtained from linear analyses were larger than 2.5% limit of NBCC (2005), while only those obtained from nonlinear analyses conducted with intensified Nisqually (2001) record were within the limits. The maximum interstorey drift ratios of FRP reinforced structures obtained from both linear and nonlinear analyses were larger than the limit of NBCC (2005).

In steel reinforced structures, both beams and columns experienced nonlinear deformations. Ductility ratios were larger when analyses were conducted with intensified artificially generated records compared to intensified Nisqually (2001) record. The maximum ductility ratios for beams were 7.3, 7.3, and 4.8 and the maximum ductility ratios for columns were 5.6, 6.6 and 3.5 when 'Short Event 4', 'Long Event 1' and Nisqually (2001) records, intensified by 100%, were used respectively, and anchorage slip effect was not considered. These ratios were 4.2, 5.5 and 2.3 for beams and 4.0, 6.1 and 2.1 for columns, when anchorage slip effect was considered.

FRP reinforced structures experienced some form of failure when the intensified earthquake records were used. The intensified Short Event 4 Earthquake Record resulted in the failure of all the beams on the first three stories through the crushing of concrete which was followed by the rupturing of FRP, when anchorage slip was ignored in analysis. Some of the first storey columns also failed due to the crushing of concrete. When anchorage slip was considered in analysis, only one of the first-storey beams experienced failure due to concrete crushing. The intensified Long Event 1 Earthquake Record produced the crushing of concrete followed by the rupture of FRP in all the beams of the first two stories, one of the third-storey beam and all first-storey columns when anchorage slip was ignored. When anchorage slip was considered, all the beams on the first three stories and some of the columns on the first storey failed due to the crushing of concrete, followed by the rupture of FRP. The intensified Nisqually 2001 record resulted in the crushing of concrete and subsequent rupturing of FRP in all the beams on the first three stories, as well as the columns on the first storey when anchorage slip was ignored. When anchorage slip was incorporated in analysis, some of the beams on the first three stories failed due to concrete crushing followed by FRP rupture, while no failure was observed in the columns.

Sample first-storey beam and column moment-chord rotation hysteretic relationships are illustrated in Figs 4.88 and 4.99 for steel and FRP reinforced buildings, with and without anchorage slip. The results indicate that the steel-reinforced concrete building developed significant reductions in seismic force demands as a result of inelastic response. This reduced force demands were accompanied with increased ductility demands as expected. FRP reinforced concrete buildings experienced a small reduction in force demands due to the inelasticity of columns, while maintaining approximately the same deformation demands. The intensity of earthquake motion used was too severe for the FRP reinforced concrete building, and many elements experienced failure. However, the results do indicate that well confined columns can develop some limited ductility, even though the beams experience brittle failure beyond their elastic limits, without any significant inelastic deformability.

Table 4.1 Comparisons of base shears obtained using equivalent static load approach of NBCC-2005 and dynamic analysis of SEQUAKE

		STEEL REINFORCED BUILDINGS			FRP REINFORCED BUILDINGS		
	Number of Storeys	$V_{NBCC-2005}$ (kN)	$V_{SEQUAKE}$ (kN) <i>Artificially Generated EQ</i>	$V_{SEQUAKE}$ (kN) <i>Previously Recorded EQ</i>	$V_{NBCC-2005}$ (kN)	$V_{SEQUAKE}$ (kN) <i>Artificially Generated EQ</i>	$V_{SEQUAKE}$ (kN) <i>Previously Recorded EQ</i>
Ottawa	5	568	541		1934	532	
	10	588	446		1998	456	
	15	635	442		2153	453	
Vancouver	5	728	1804	1818	4950	3478	4370
	10	995	1910	3115	6572	3443	4450
	15	1224	1964	3950	8322	4975	6310

Note: Dynamic analyses were conducted without the effects of anchorage slip.

Table 4.2 Comparisons of maximum interstorey drift demands obtained using equivalent static load approach of NBCC-2005 and dynamic analysis of SEQUAKE

		STEEL REINFORCED BUILDINGS				FRP REINFORCED BUILDINGS		
	Number of Storey	Drift Ratio (%)		Drift Ratio (%) <i>Artificially Generated EQ</i>	Drift Ratio (%) <i>Previously Recorded EQ</i>	Drift Ratio (%)	Drift Ratio (%) <i>Artificially Generated EQ</i>	Drift Ratio (%) <i>Previously Recorded EQ</i>
		Elastic	Inelastic (Elastic x R _d R _o)					
Ottawa	5	0.4	1.36	0.37		1.26	0.4	
	10	0.4	1.36	0.24		1.47	0.24	
	15	0.32	1.08	0.18		1.37	0.19	
Vancouver	5	0.3	2.04	1.48	1.39	1.9	1.41	1.94
	10	0.32	2.2	1.33	1.07	1.56	0.95	0.62
	15	0.34	2.3	0.79	1.17	2.12	1.13	1.25

Note: Dynamic analyses were conducted without the effects of anchorage slip.

Table 4.3 Effects of the incorporation of anchorage slip in dynamic inelastic response history analysis of 5 storey buildings in Vancouver on their base shear

Vancouver		STEEL REINFORCED BUILDINGS			FRP REINFORCED BUILDINGS		
		$V_{NBCC-2005}$ (kN)	$V_{SEQUAKE}$ (kN) Without anchorage slip effect	$V_{SEQUAKE}$ (kN) With anchorage slip effect	$V_{NBCC-2005}$ (kN)	$V_{SEQUAKE}$ (kN) Without anchorage slip effect	$V_{SEQUAKE}$ (kN) With anchorage slip effect
<i>Artificially Generated EQ</i>	Short Event 4	728	1804	1365	4950	3478	3034
	Long Event 1	728	1754	1776	4950	3660	4120
<i>Previously Recorded EQ</i>	Nisqually (2001)	728	1818	1553	4950	4370	3368

Table 4.4 Effects of the incorporation of anchorage slip in dynamic inelastic response history analysis of 5 storey buildings in Vancouver on maximum interstorey drift ratios

Vancouver Maximum Interstorey Drift Ratios (%)		STEEL REINFORCED BUILDINGS				FRP REINFORCED BUILDINGS		
		Elastic (design force level) (%)	Inelastic (Elastic x R) (%)	Without anchorage slip effect (%)	With anchorage slip effect (%)	Elastic (design force level) (%)	Without anchorage slip effect (%)	With anchorage slip effect (%)
Artificially Generated EQ	Short Event 4	0.3	2.04	1.48	1.66	1.9	1.41	1.51
	Long Event 1	0.3	2.04	1.46	1.97	1.9	1.26	2.05
Previously Recorded EQ	Nisqually (2001)	0.3	2.04	1.39	1.4	1.9	1.94	1.67

Table 4.5 Maximum base shears for 5 storey buildings in Vancouver when analyzed under intensified earthquake motions

Vancouver		STEEL REINFORCED BUILDINGS				FRP REINFORCED BUILDINGS			
		V (kN) Elastic Dynamic Analysis		V (kN) Inelastic Dynamic Analysis		V (kN) Elastic Dynamic Analysis		V (kN) Inelastic Dynamic Analysis	
		Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect
<i>Artificially Generated EQ intensified by 100%</i>	Short Event 4	7192	4140	2775	2208	6808	6944	5567	6506
	Long Event 1	8426	7730	2557	2536	6526	9060	7592	5874
<i>Previously Recorded EQ Intensified by 100%</i>	Nisqually (2001)	9632	5208	2645	3312	9870	7660	4925	5998

Table 4.6 Maximum interstorey drift ratios for 5 storey buildings in Vancouver when analyzed under intensified earthquake motions

Vancouver		STEEL REINFORCED BUILDINGS				FRP REINFORCED BUILDINGS			
		Elastic Dynamic Analysis		Inelastic Dynamic Analysis		Elastic Dynamic Analysis		Inelastic Dynamic Analysis	
		Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect	Without anchorage slip effect	With anchorage slip effect
<i>Artificially Generated EQ intensified by 100%</i>	Short Event 4	3.03	3.14	2.97	3.22	2.84	2.94	3.36	2.94
	Long Event 1	3.39	5.48	2.68	3.69	2.51	4.03	5.27	5.58
<i>Previously Recorded EQ Intensified by 100%</i>	Nisqually (2001)	4.16	3.3	2.09	1.8	3.92	3.69	3.71	3.99

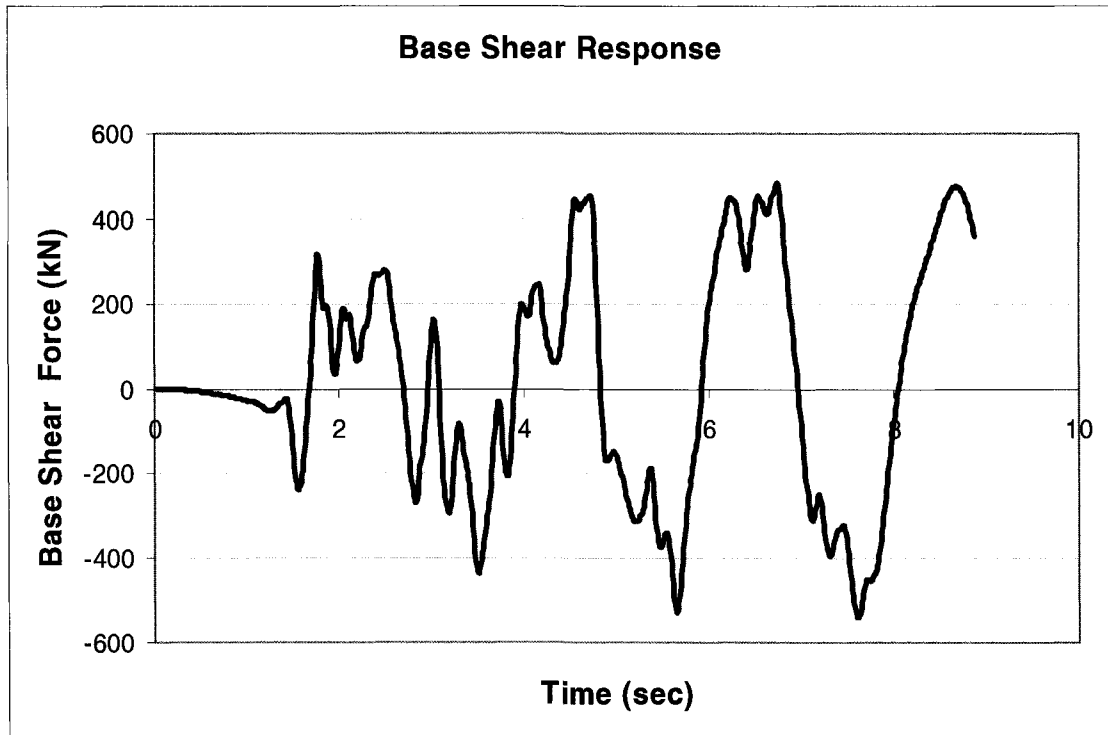


Figure 4.1 Base shear response of the 5 storey steel reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

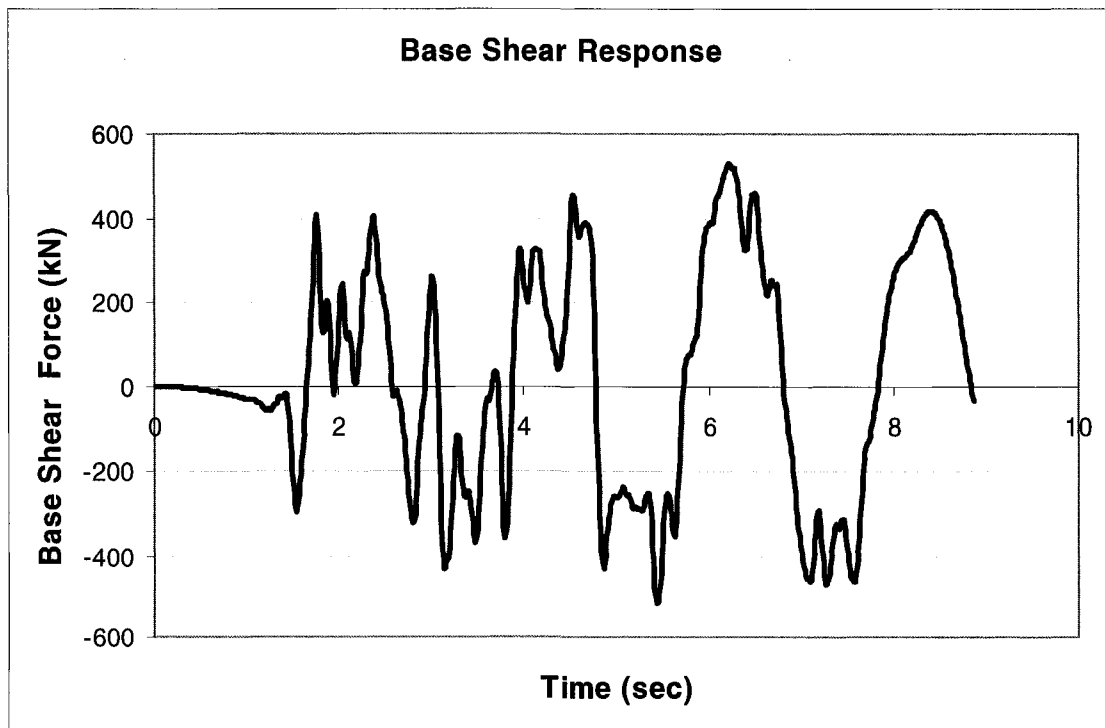


Figure 4.2 Base shear response of the 5 storey FRP reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

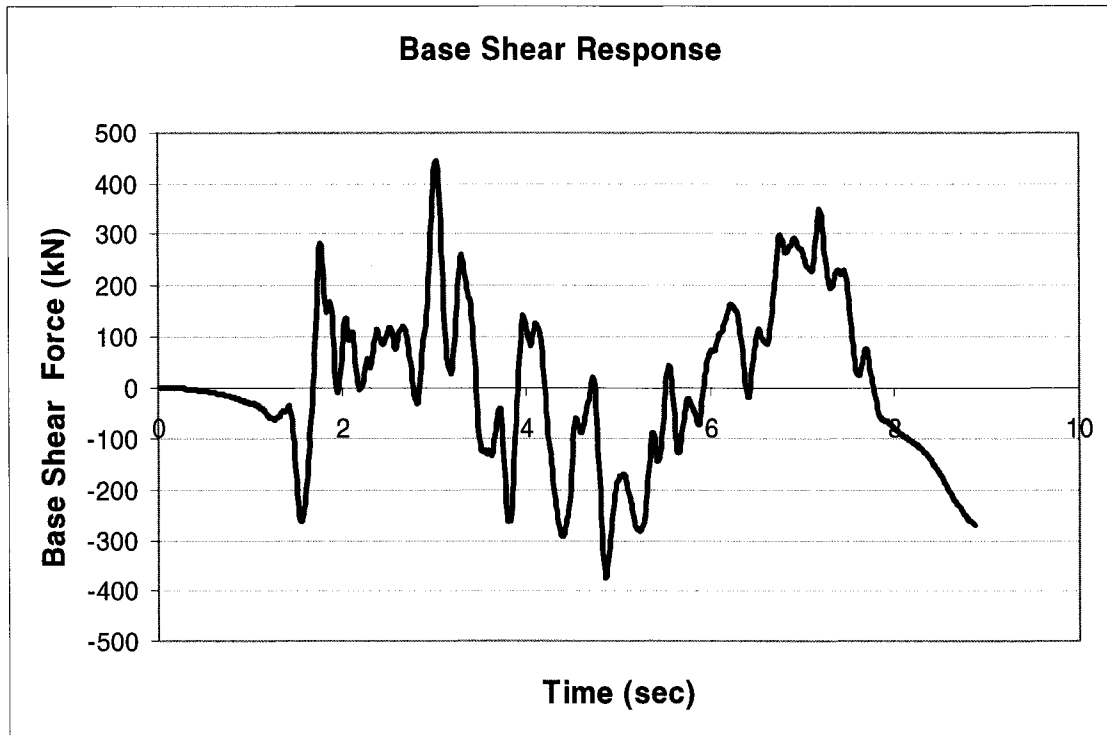


Figure 4.3 Base shear response of the 10 storey steel reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

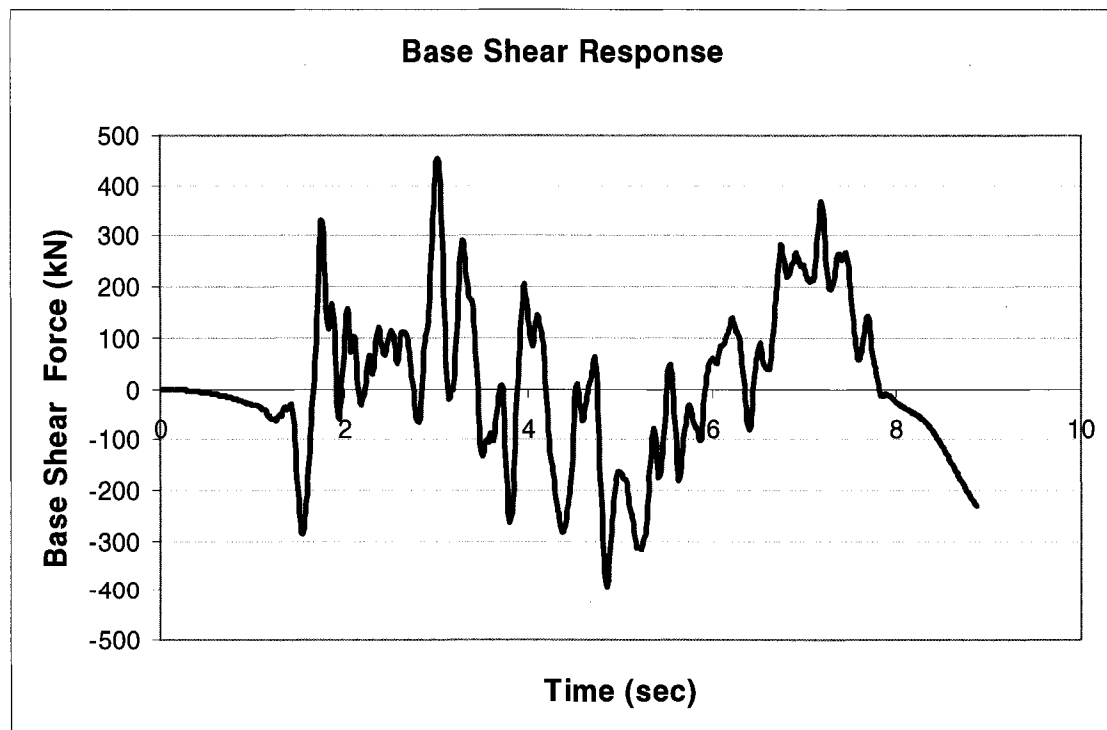


Figure 4.4 Base shear response of the 10 storey FRP reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

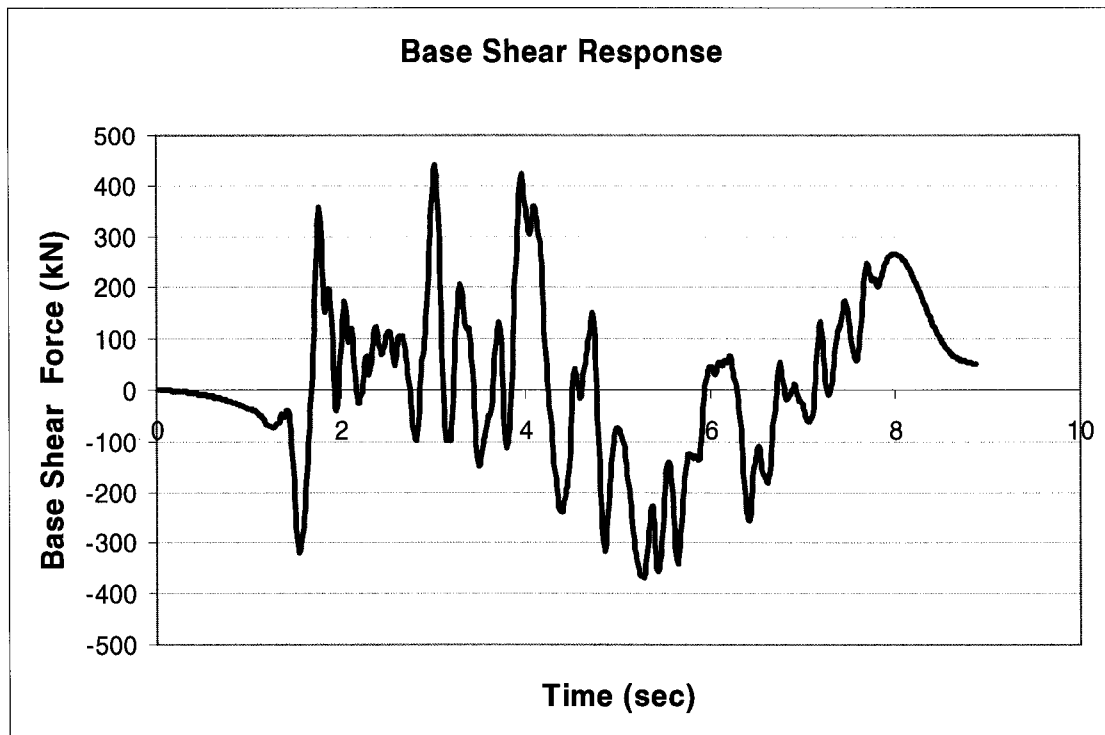


Figure 4.5 Base shear response of the 15 storey steel reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

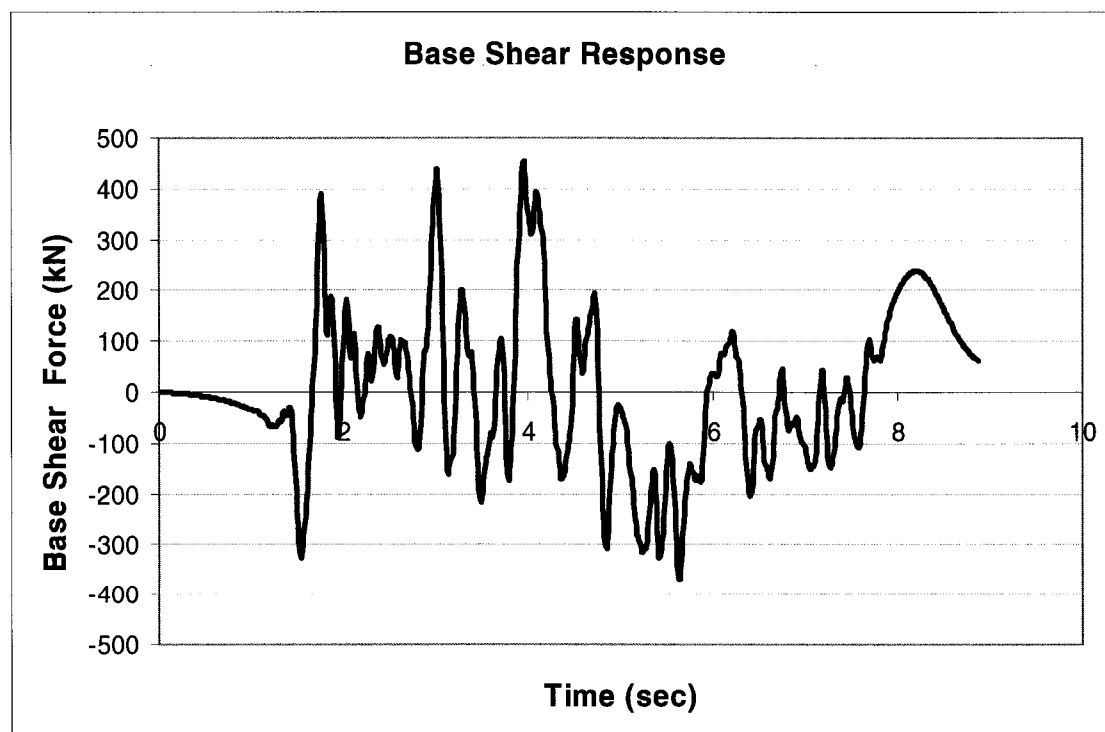


Figure 4.6 Base shear response of the 15 storey FRP reinforced building in Ottawa analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

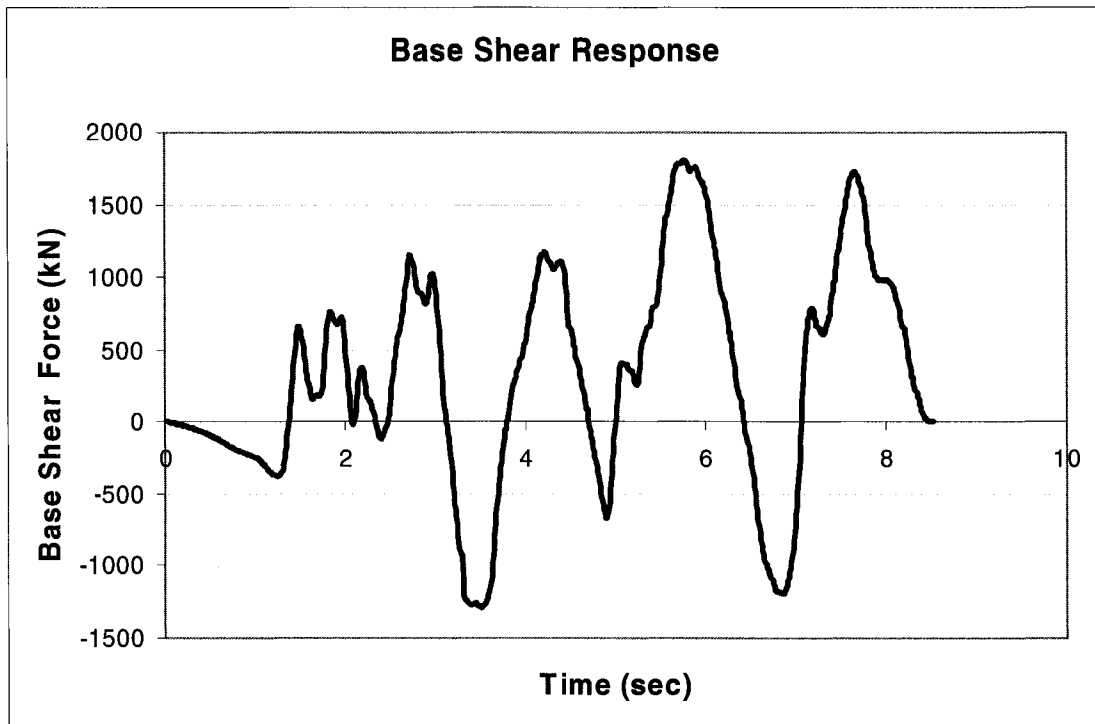


Figure 4.7 Base shear response of the 5 storey steel reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

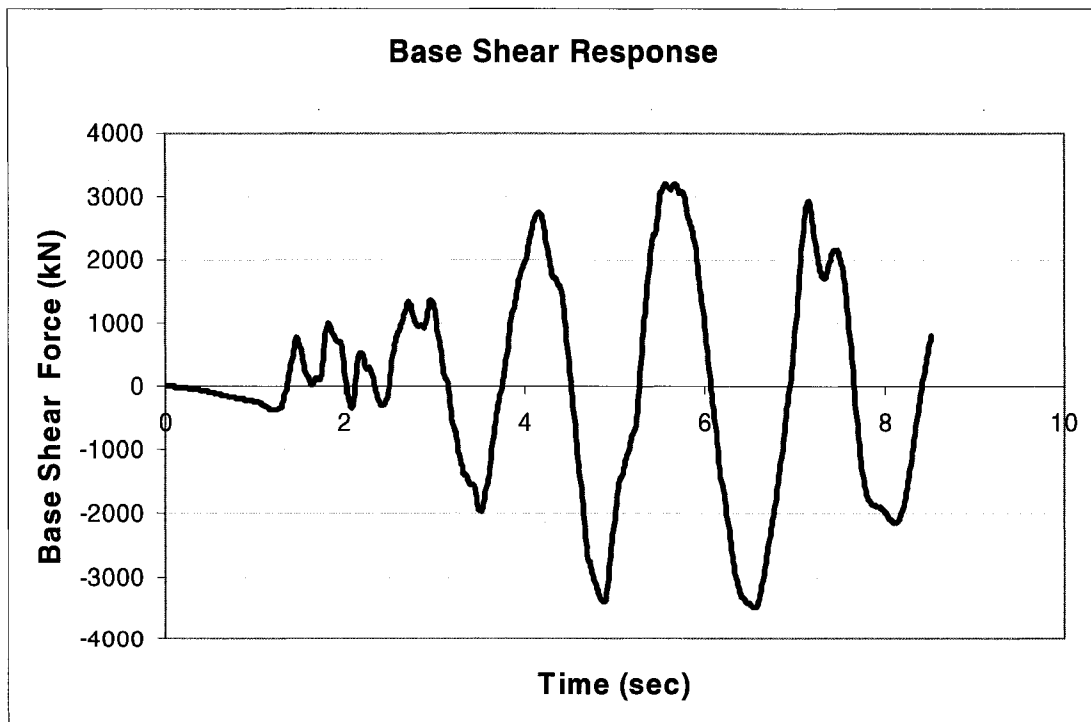


Figure 4.8 Base shear response of the 5 storey FRP reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

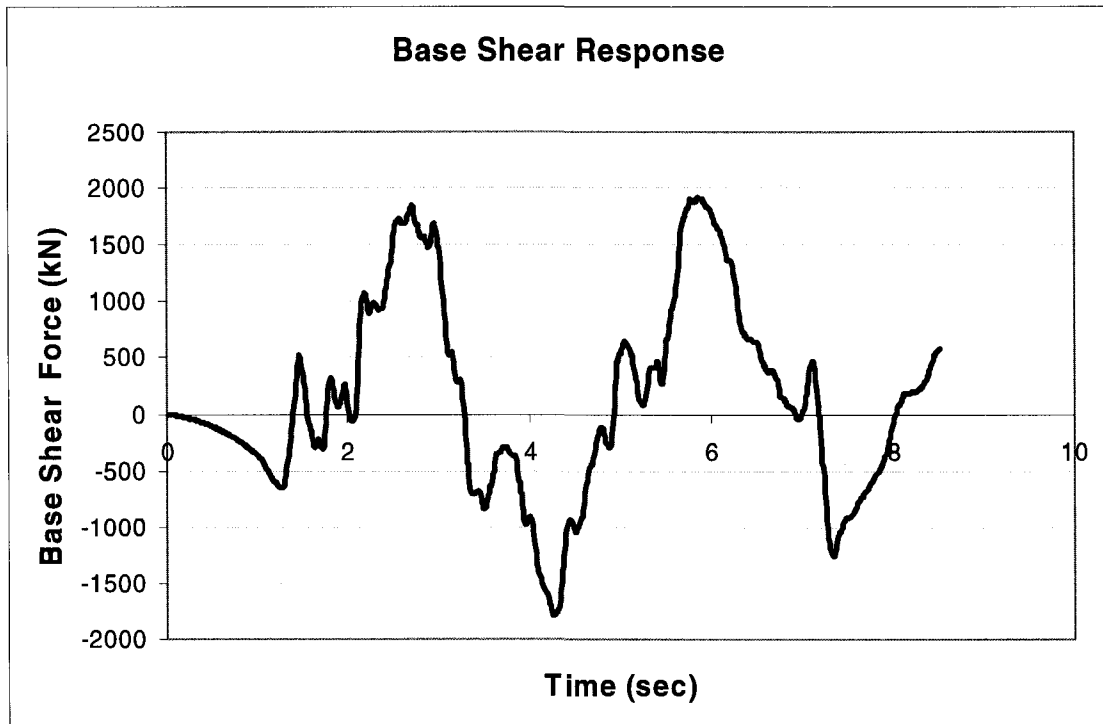


Figure 4.9 Base shear response of the 10 storey steel reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

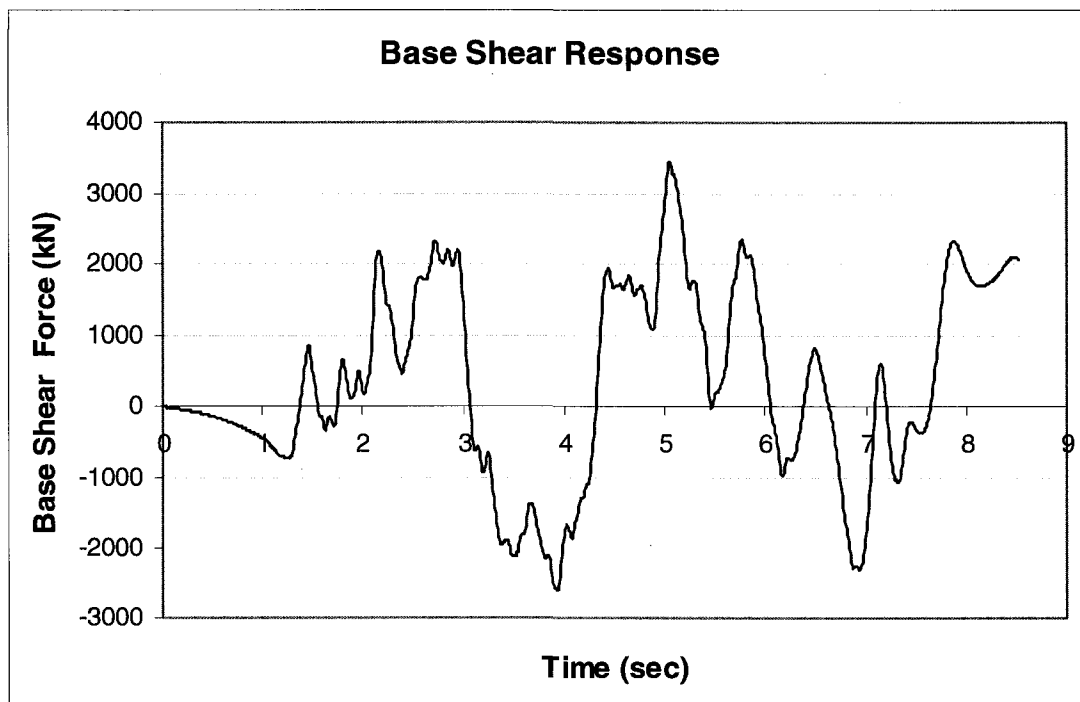


Figure 4.10 Base shear response of the 10 storey FRP reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

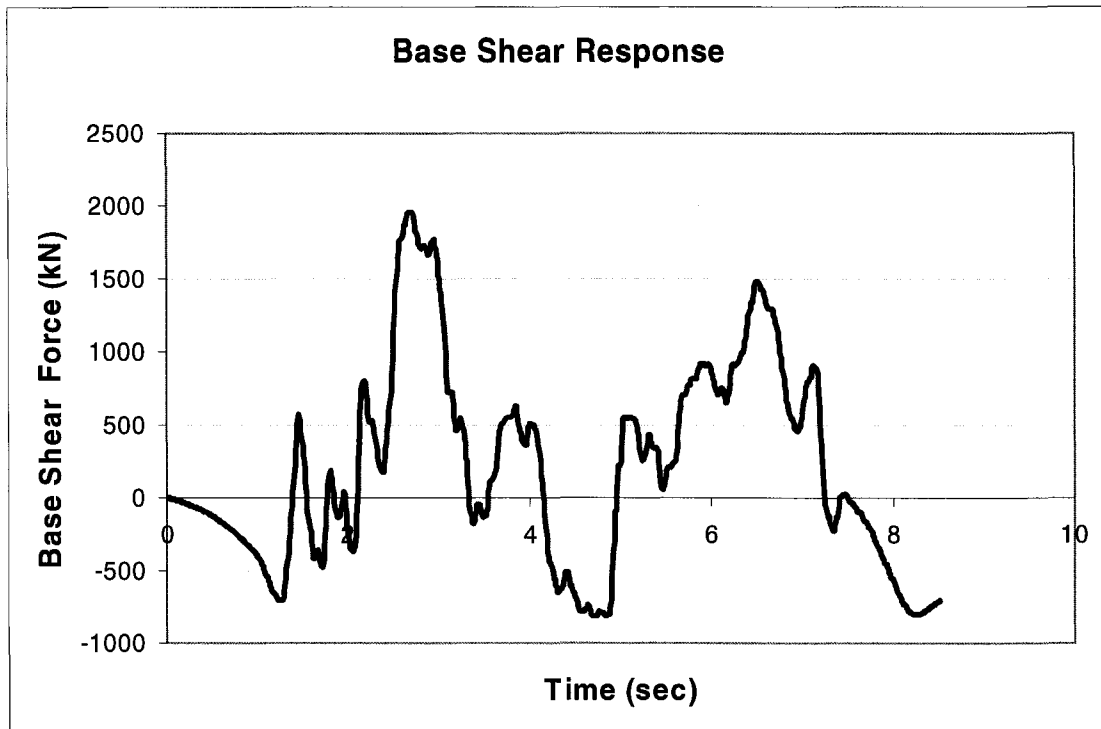


Figure 4.11 Base shear response of the 15 storey steel reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

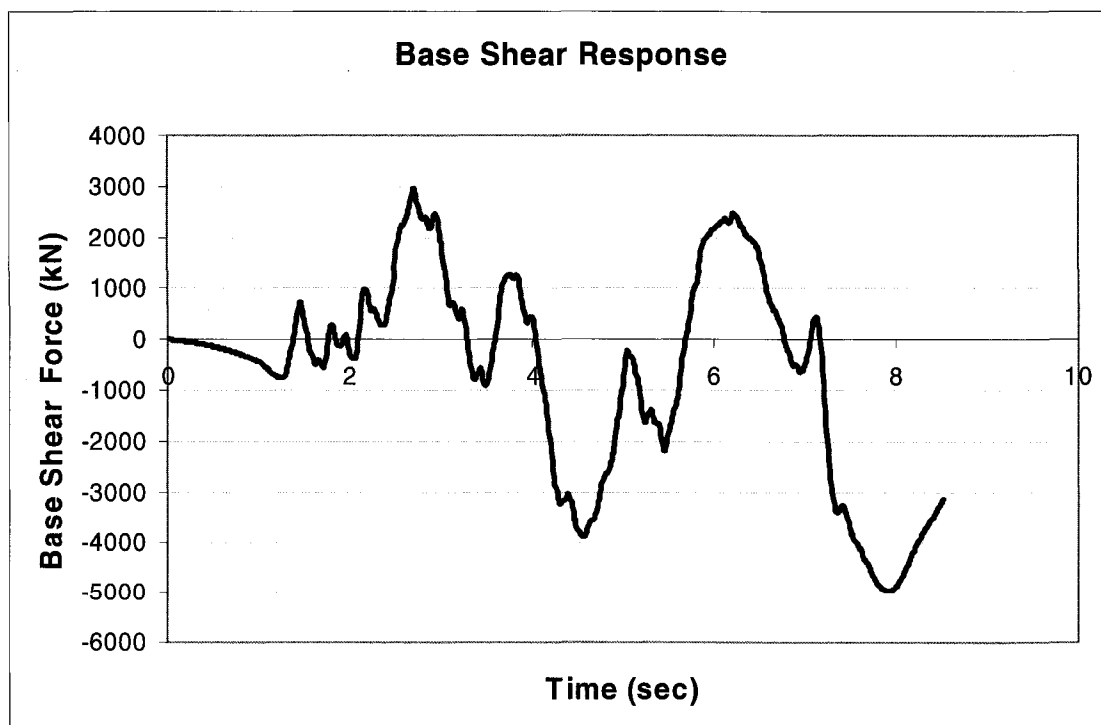


Figure 4.12 Base shear response of the 15 storey FRP reinforced building in Vancouver analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

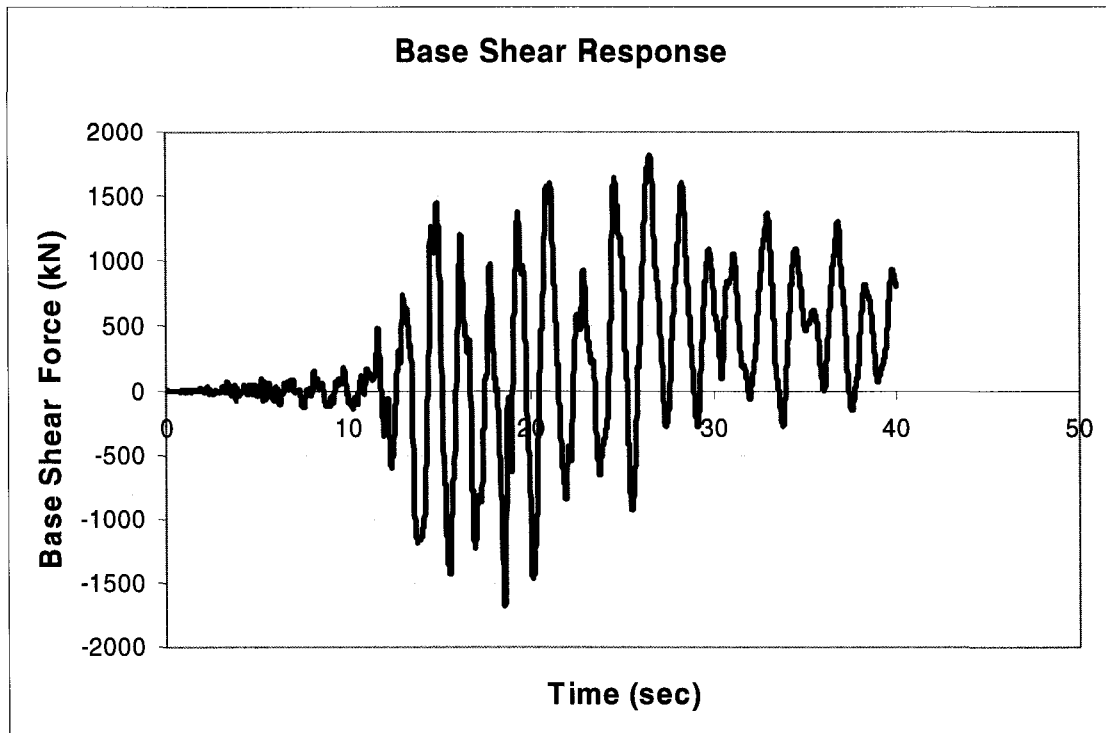


Figure 4.13 Base shear response of 5 storey steel reinforced building in Vancouver analyzed under the previously recorded earthquake Nisqually (2001)

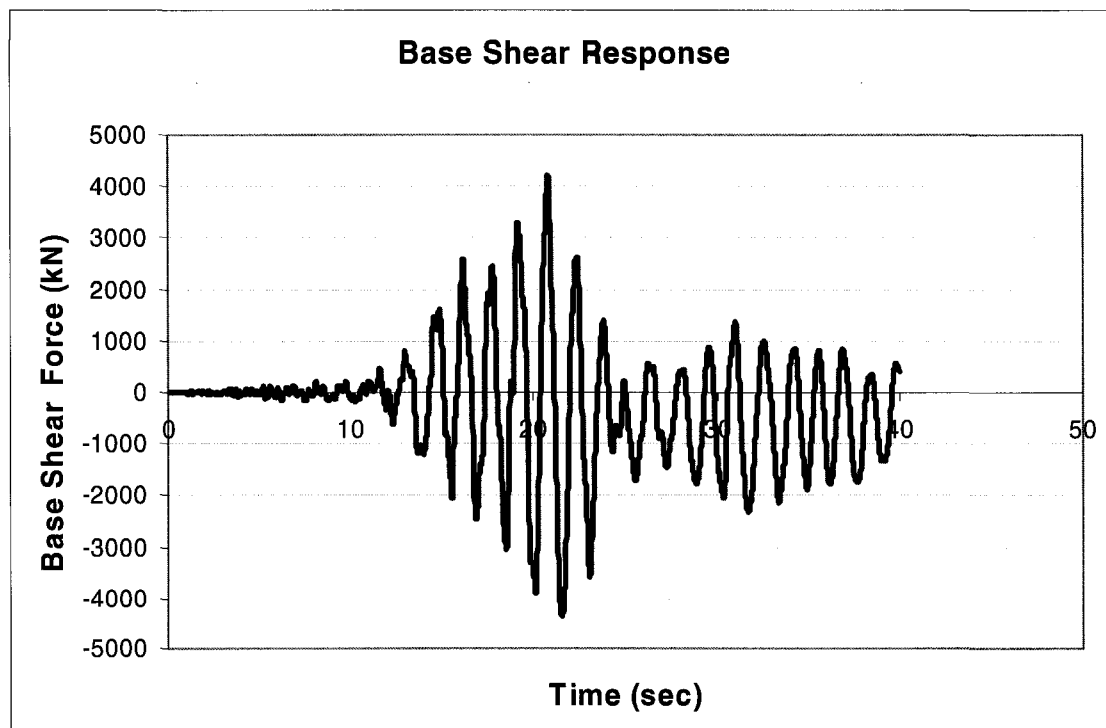


Figure 4.14 Base shear response of the 5 storey FRP reinforced building in Vancouver analyzed under the previously recorded earthquake Nisqually (2001)

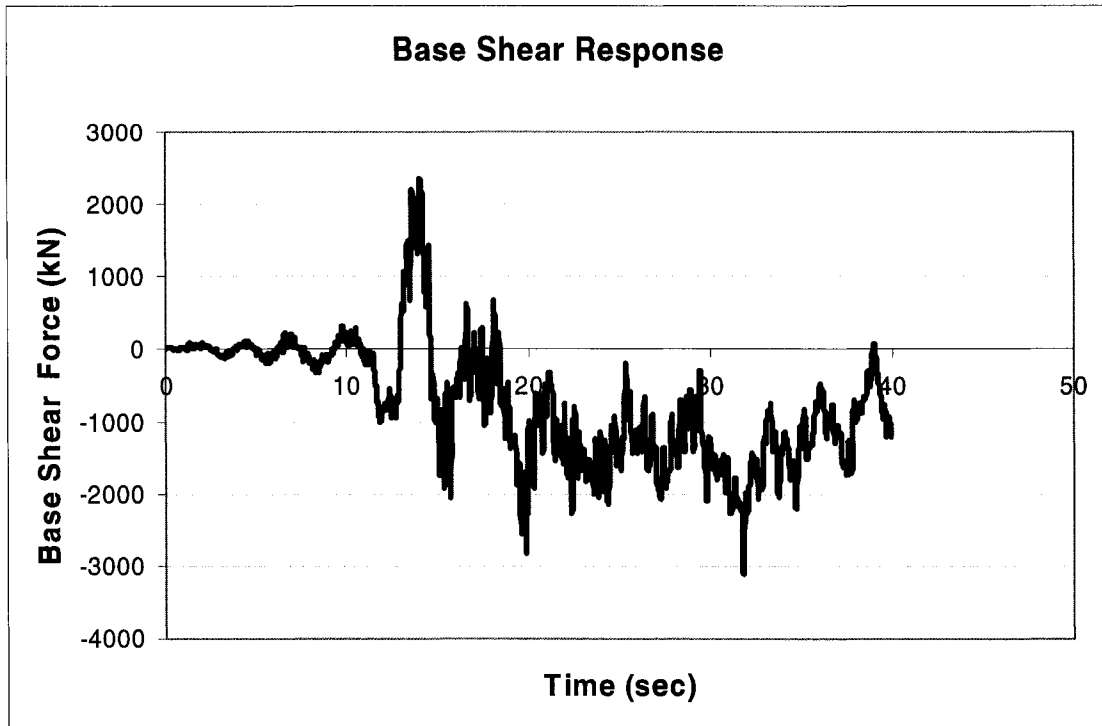


Figure 4.15 Base shear response of the 10 storey steel reinforced building in Vancouver analyzed under the previously recorded earthquake Tokachi Oki (2003)

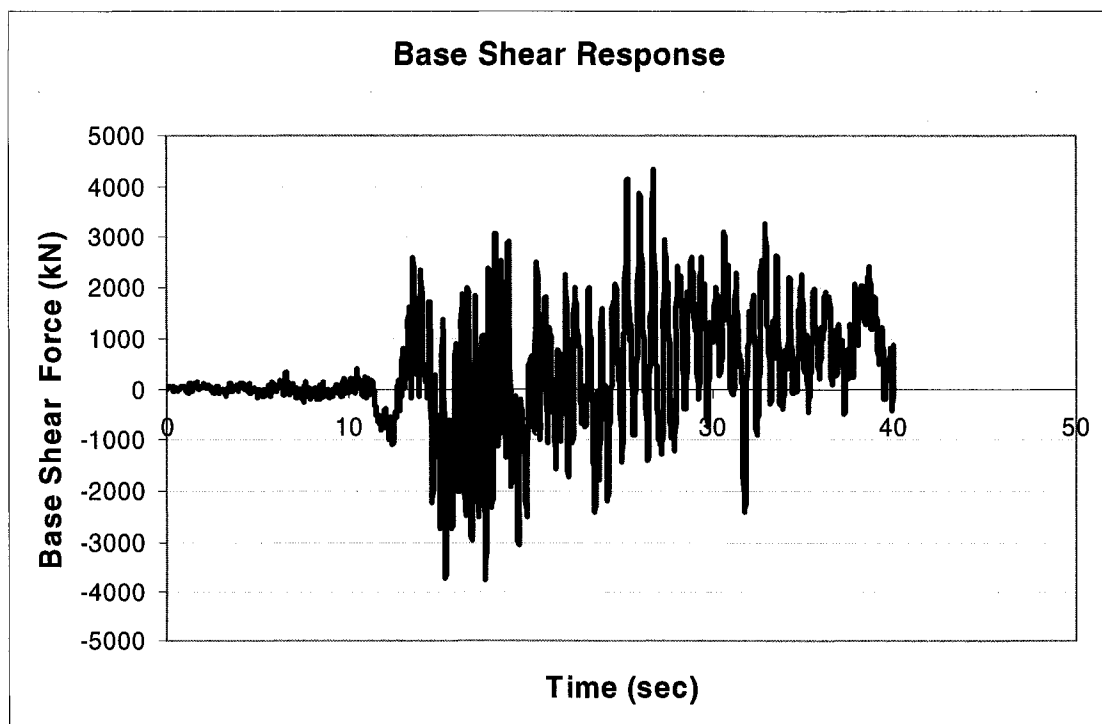


Figure 4.16 Base shear response of the 10 storey FRP reinforced building in Vancouver analyzed under the previously recorded earthquake Tokachi Oki (2003)

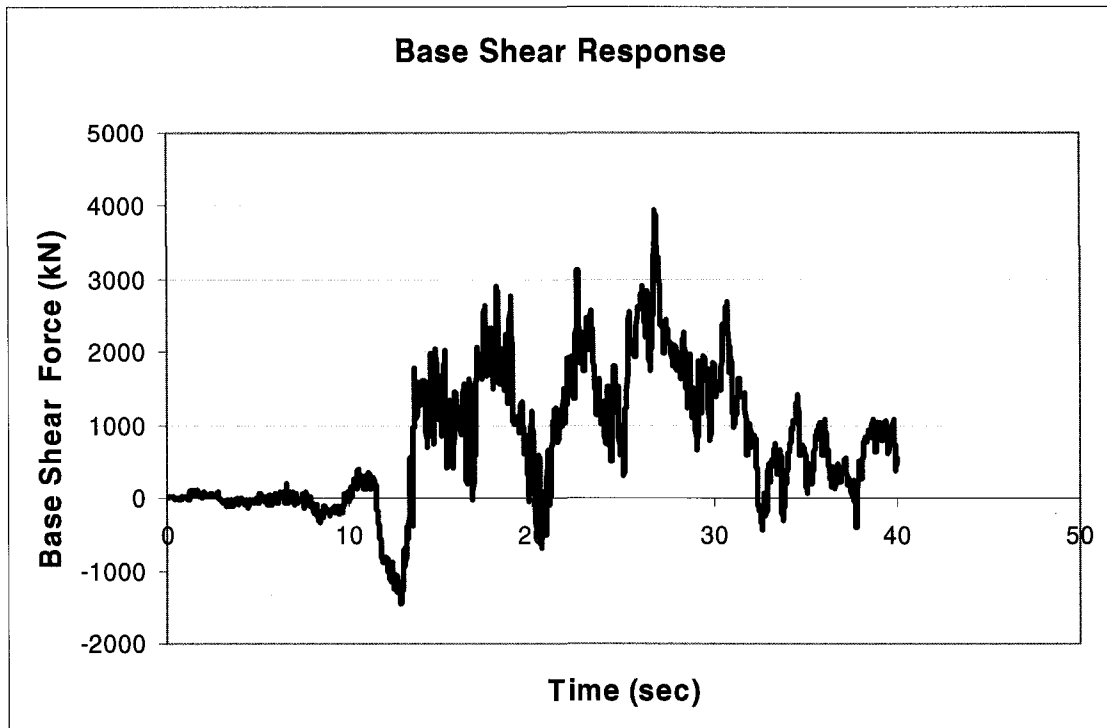


Figure 4.17 Base shear response of the 15 storey steel reinforced building in Vancouver analyzed under the previously recorded earthquake Tokachi Oki (2003)

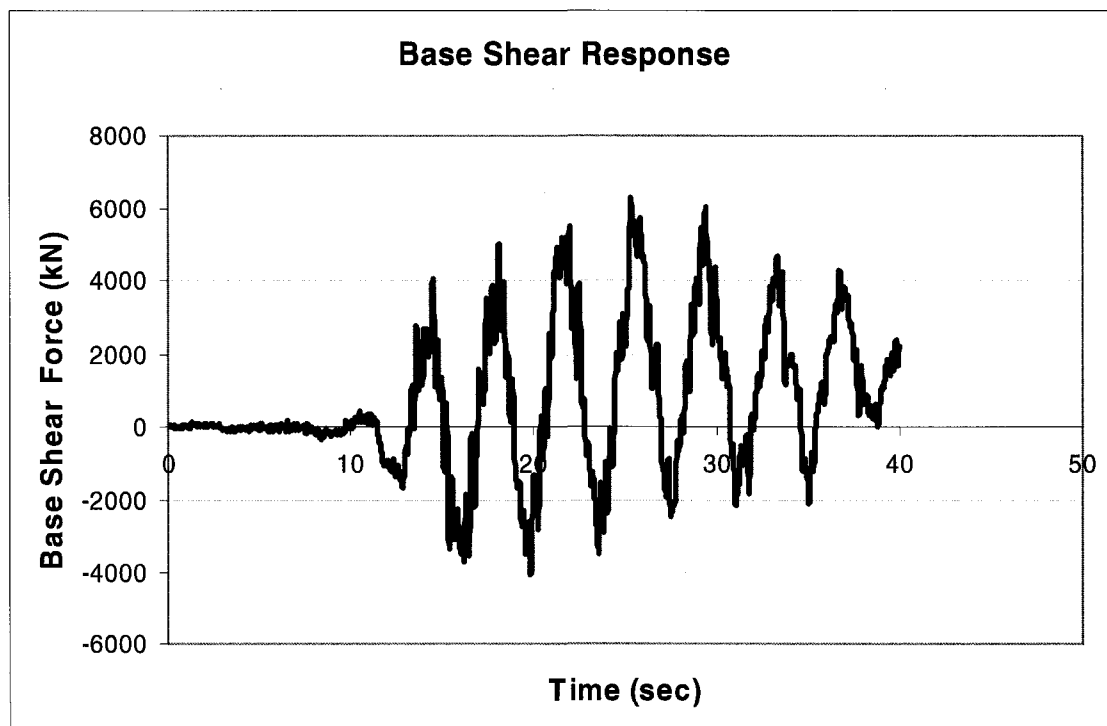


Figure 4.18 Base shear response of the 15 storey FRP reinforced building in Vancouver analyzed under the previously recorded earthquake Tokachi Oki (2003)

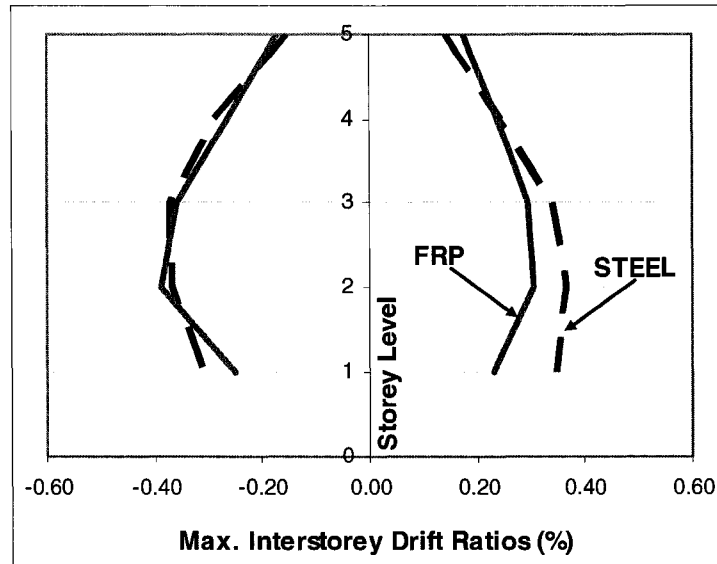


Figure 4.19 Maximum interstorey drift ratios of the 5 storey building in Ottawa reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

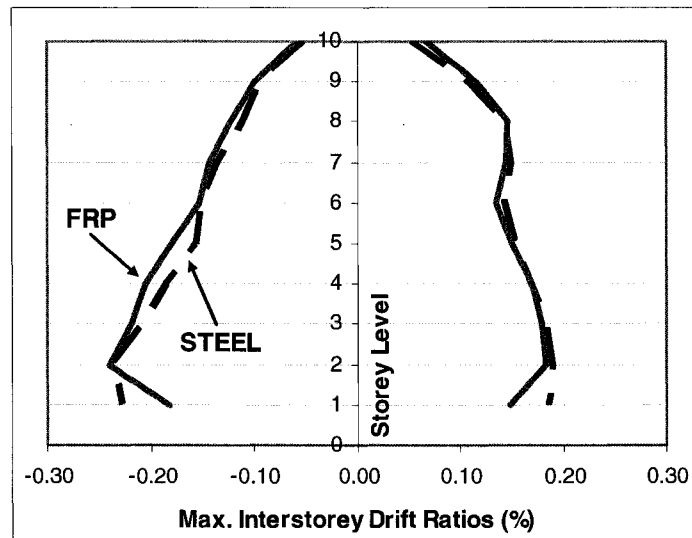


Figure 4.20 Maximum interstorey drift ratios of the 10 storey building in Ottawa reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

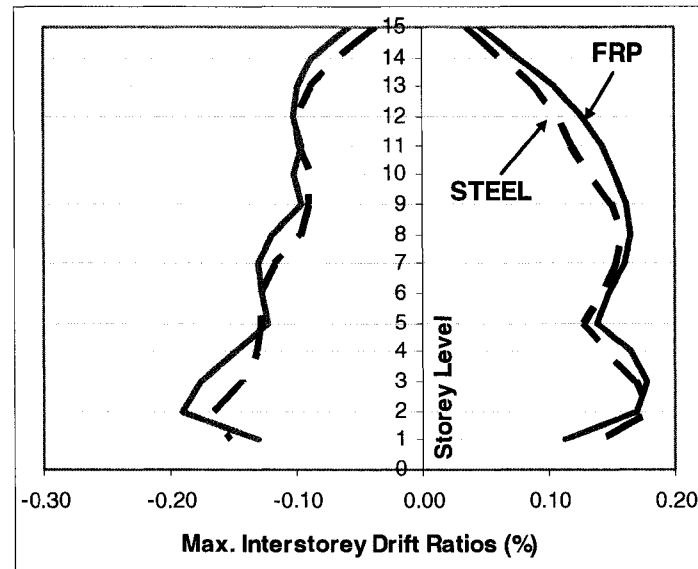


Figure 4.21 Maximum interstorey drift ratios of the 15 storey building in Ottawa reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

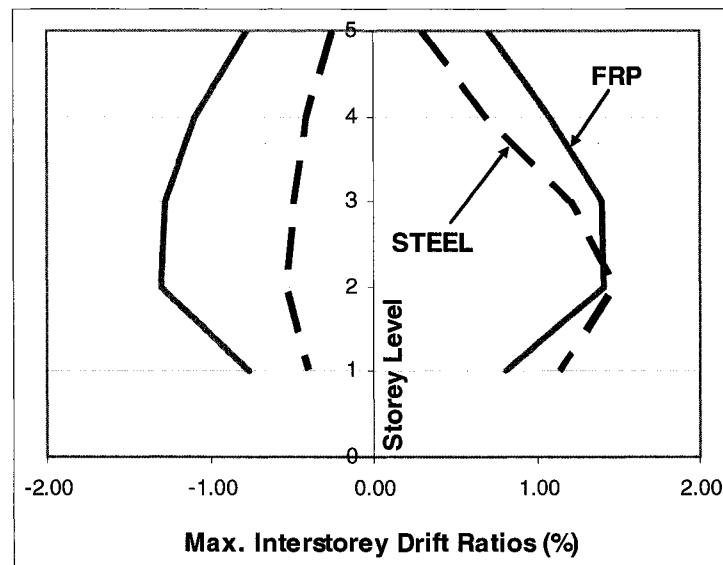


Figure 4.22 Maximum interstorey drift ratios of the 5 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

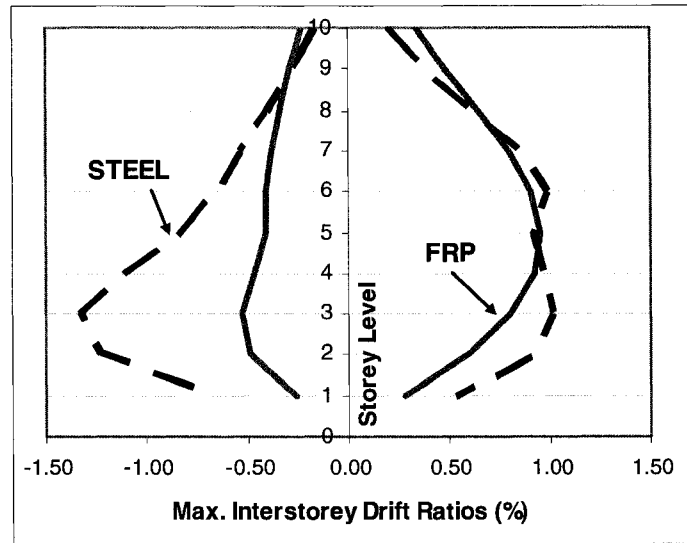


Figure 4.23 Maximum interstorey drift ratios of the 10 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

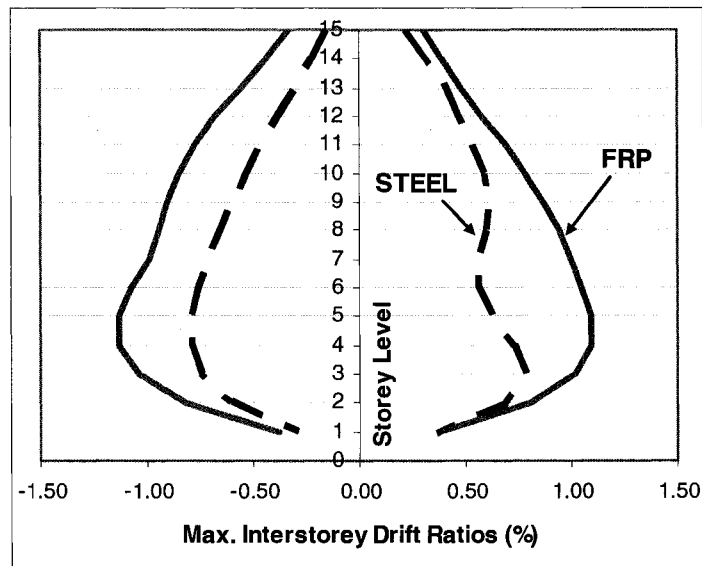


Figure 4.24 Maximum interstorey drift ratios of the 15 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

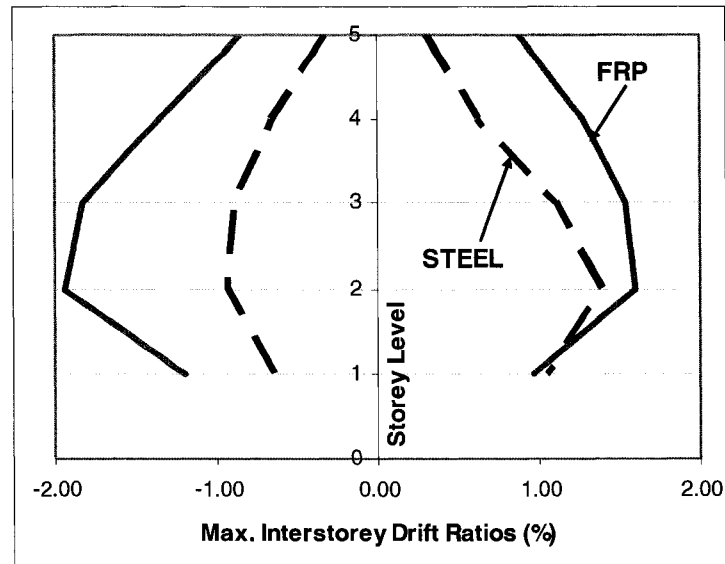


Figure 4.25 Maximum interstorey drift ratios of the 5 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the previously recorded earthquake 'Nisqually (2001)'

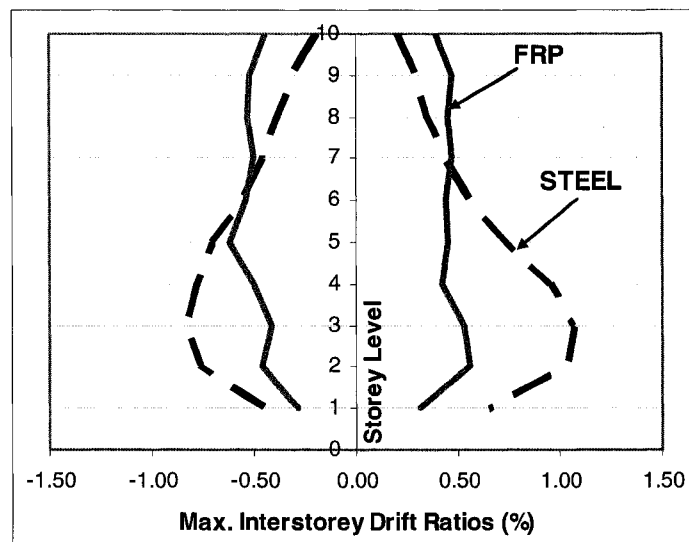


Figure 4.26 Maximum interstorey drift ratios of the 10 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

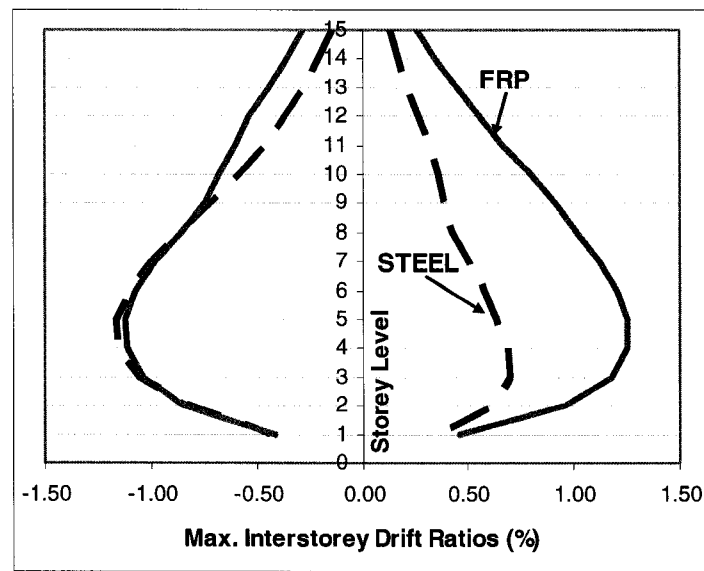


Figure 4.27 Maximum interstorey drift ratios of the 15 storey building in Vancouver reinforced with steel and FRP bars, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

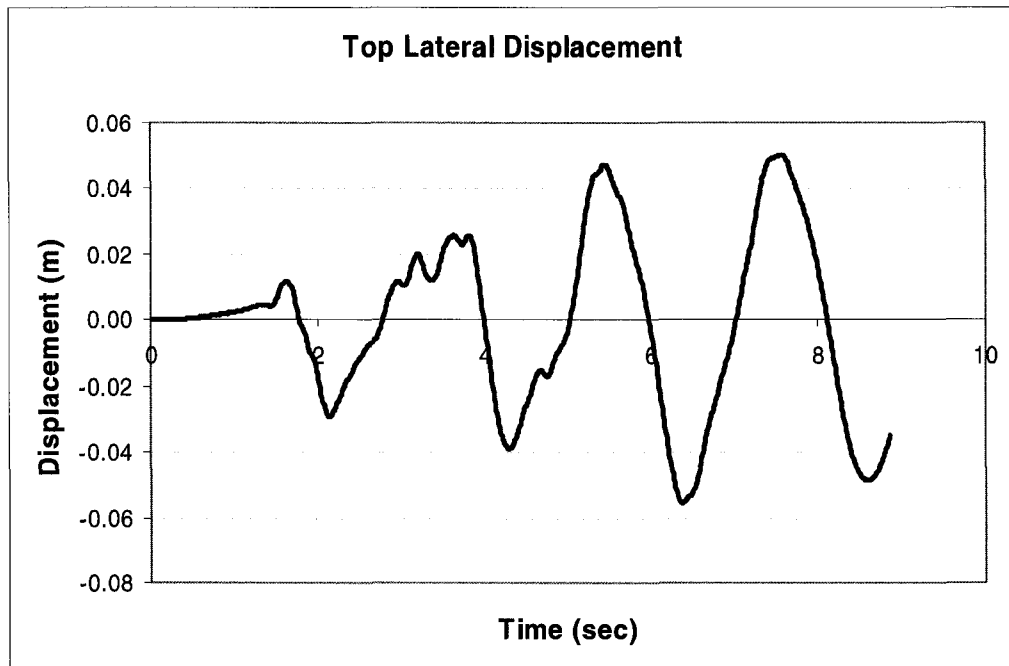


Figure 4.28 Top lateral displacement response of the 5 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

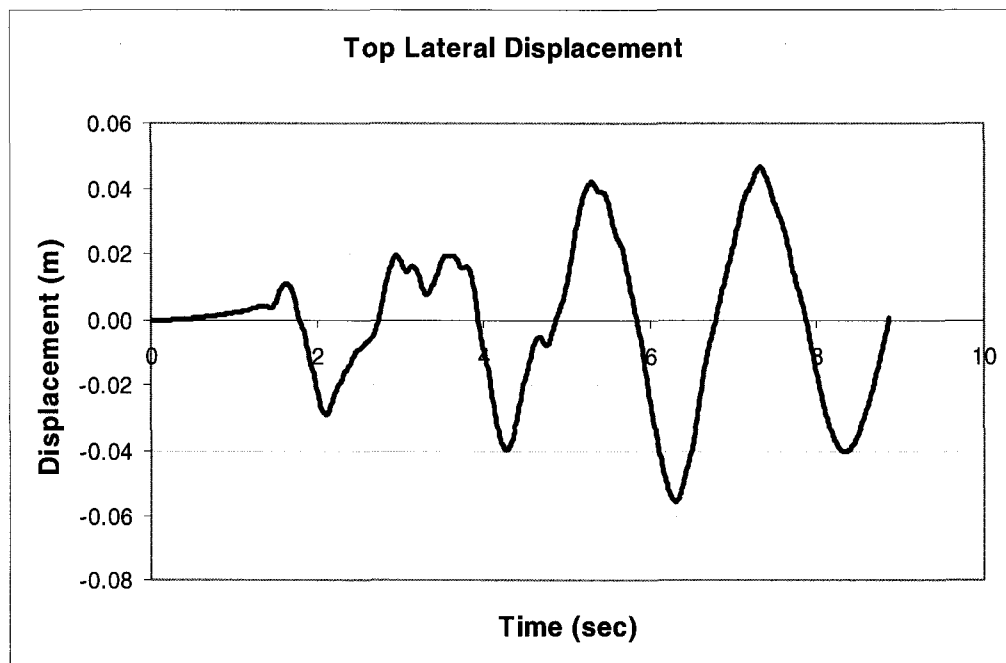


Figure 4.29 Top lateral displacement response of the 5 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

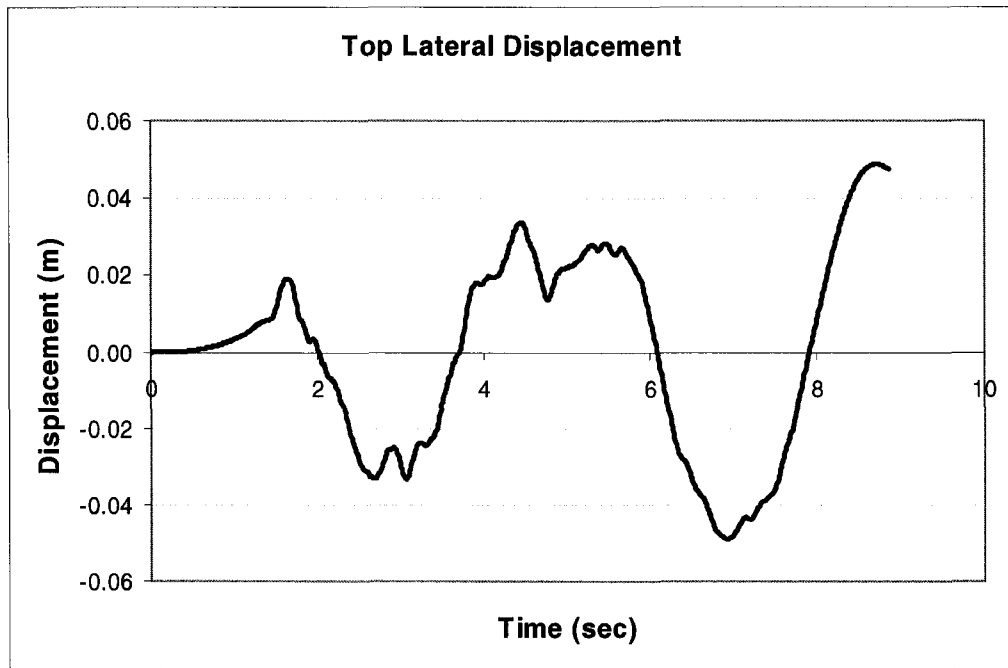


Figure 4.30 Top lateral displacement response of the 10 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

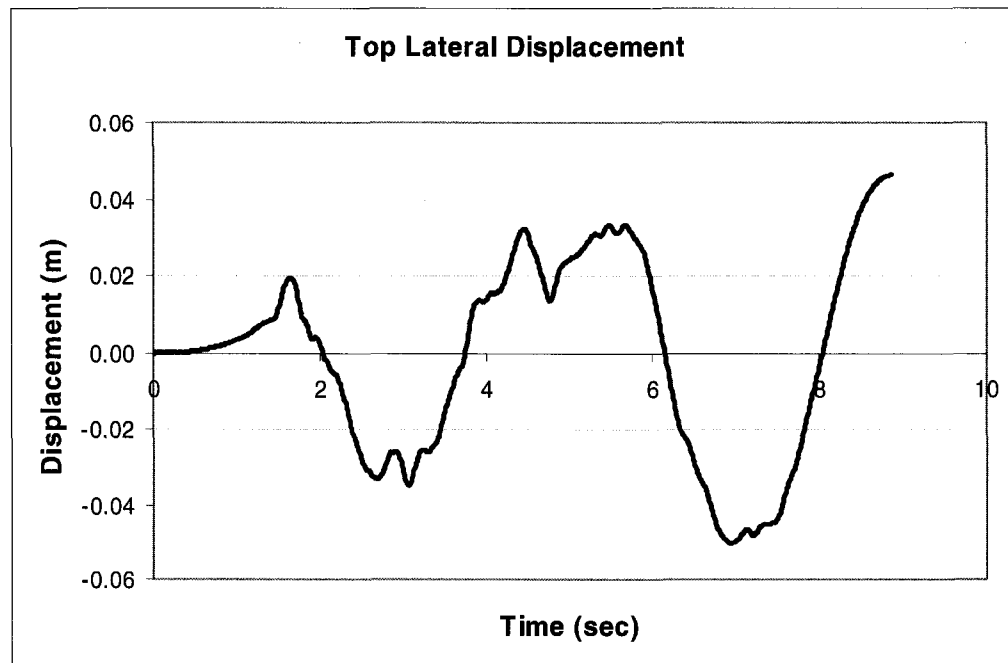


Figure 4.31 Top lateral displacement response of the 10 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

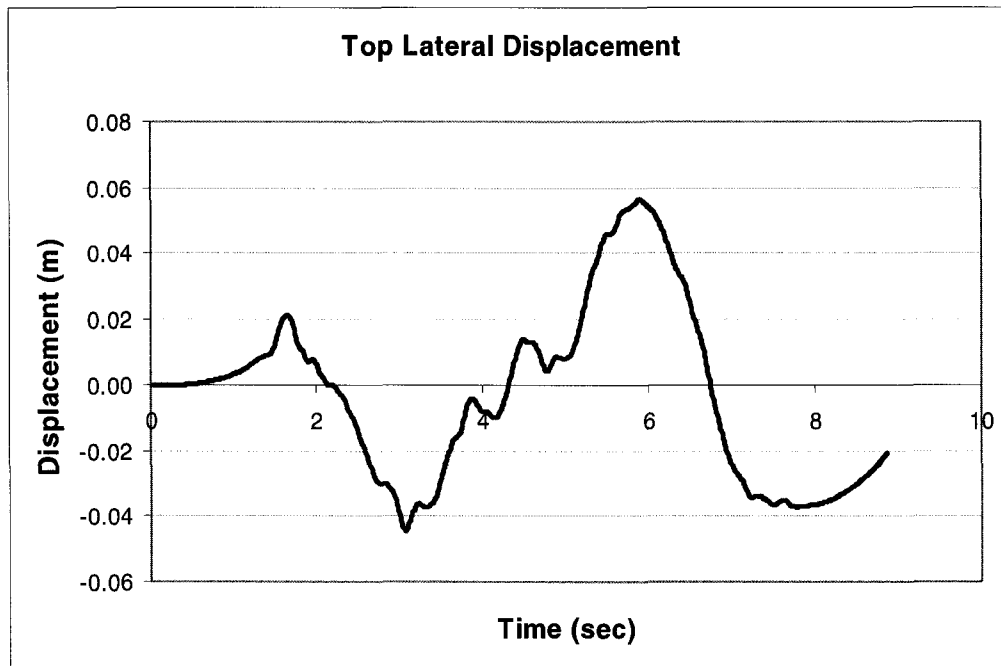


Figure 4.32 Top lateral displacement response of the 15 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

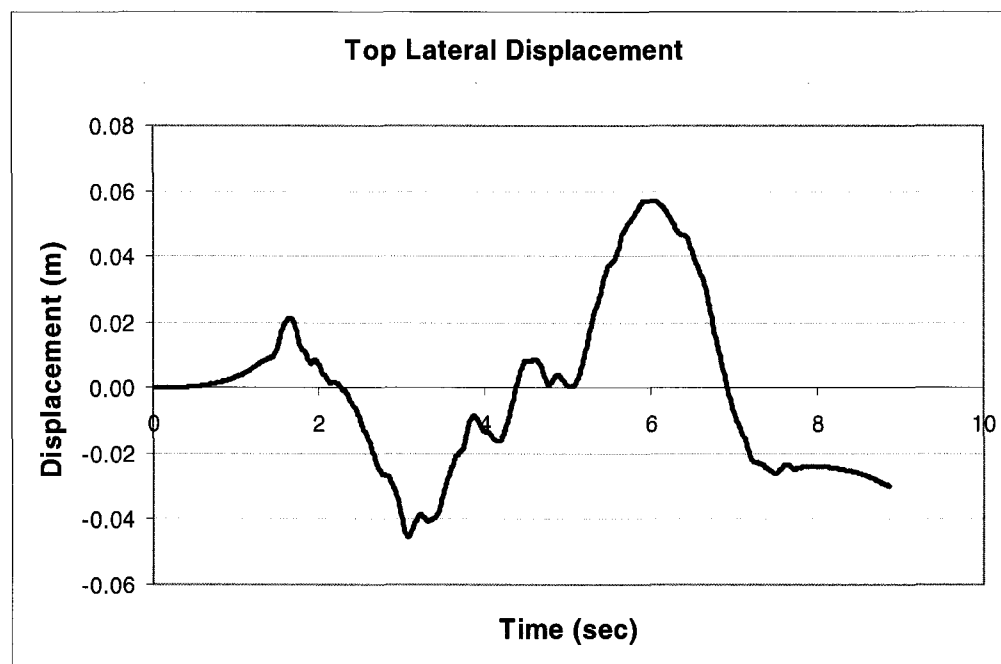


Figure 4.33 Top lateral displacement response of the 15 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

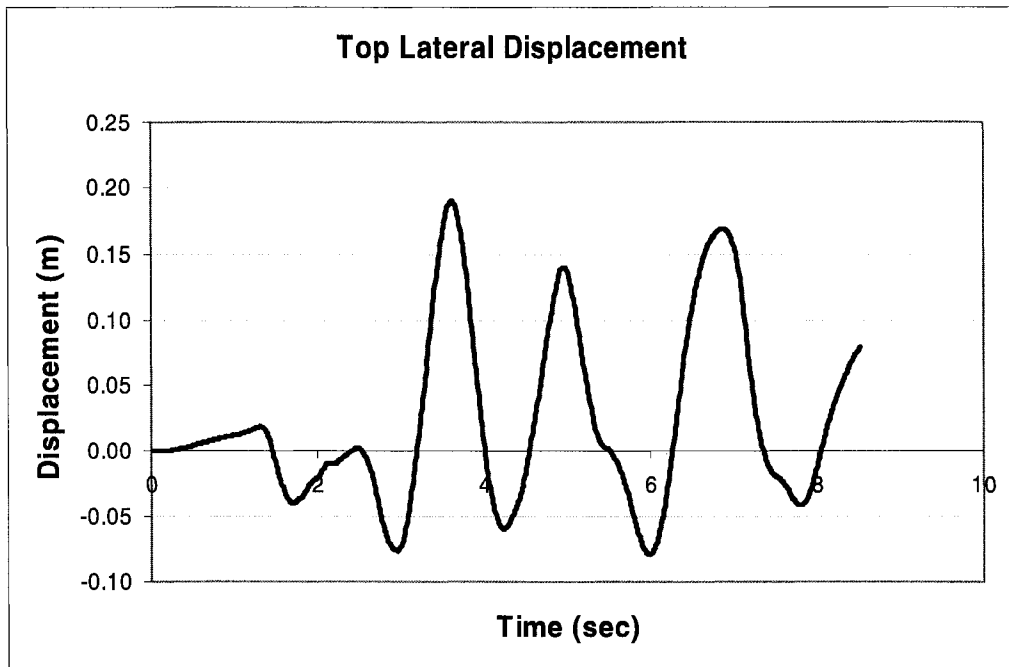


Figure 4.34 Top lateral displacement response of the 5 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

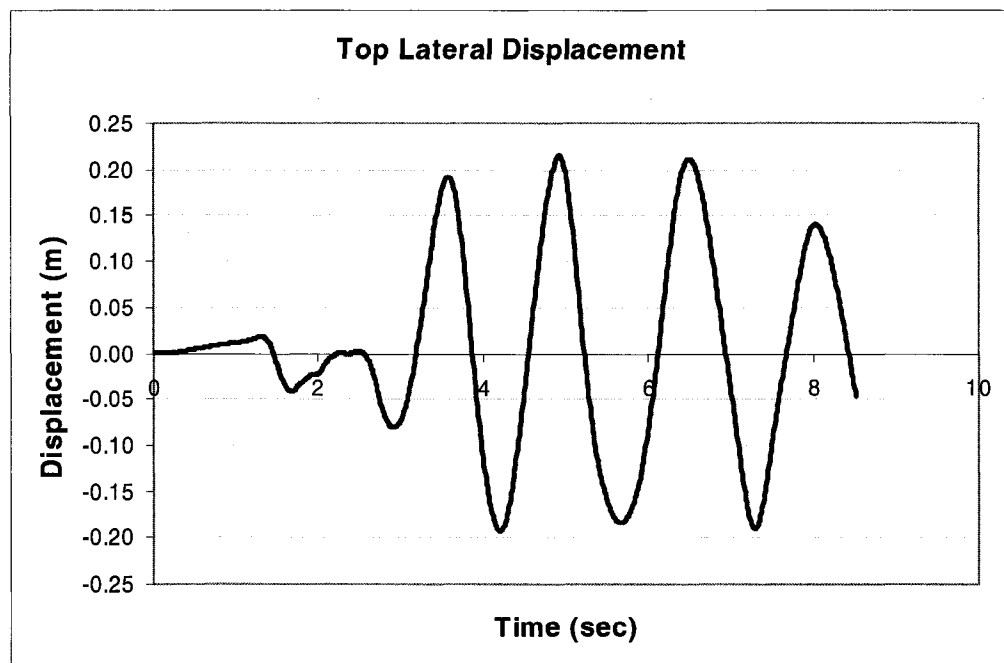


Figure 4.35 Top lateral displacement response of the 5 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

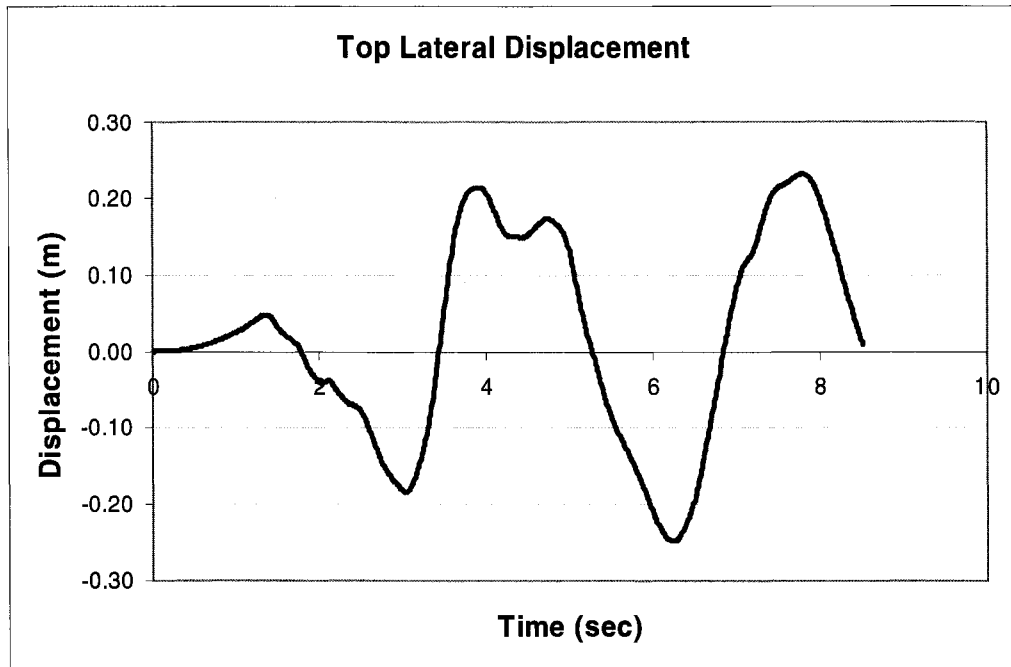


Figure 4.36 Top lateral displacement response of the 10 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

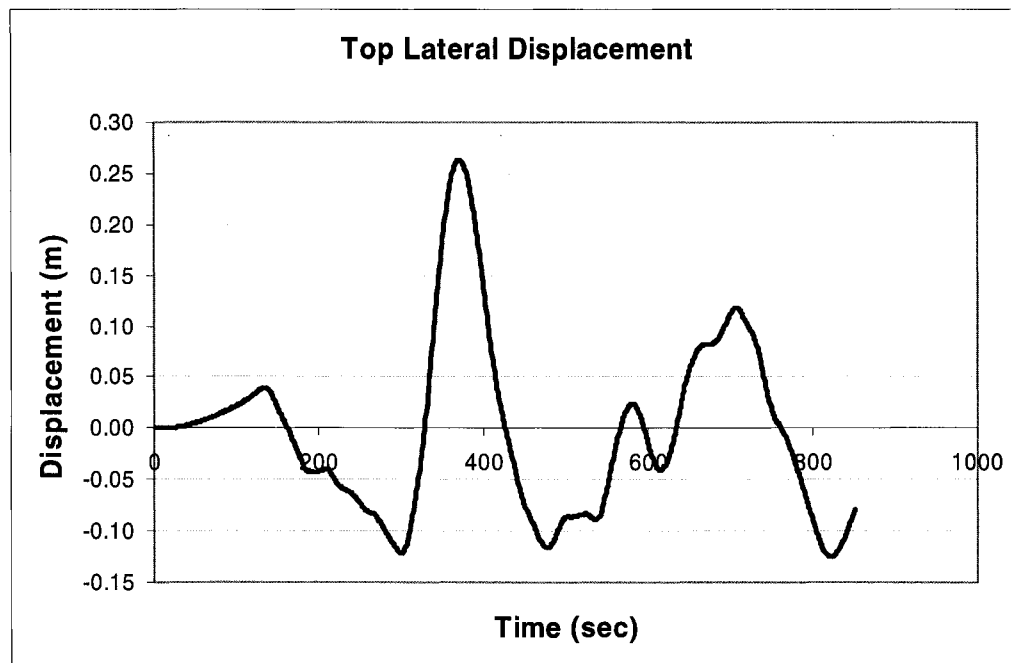


Figure 4.37 Top lateral displacement response of the 10 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

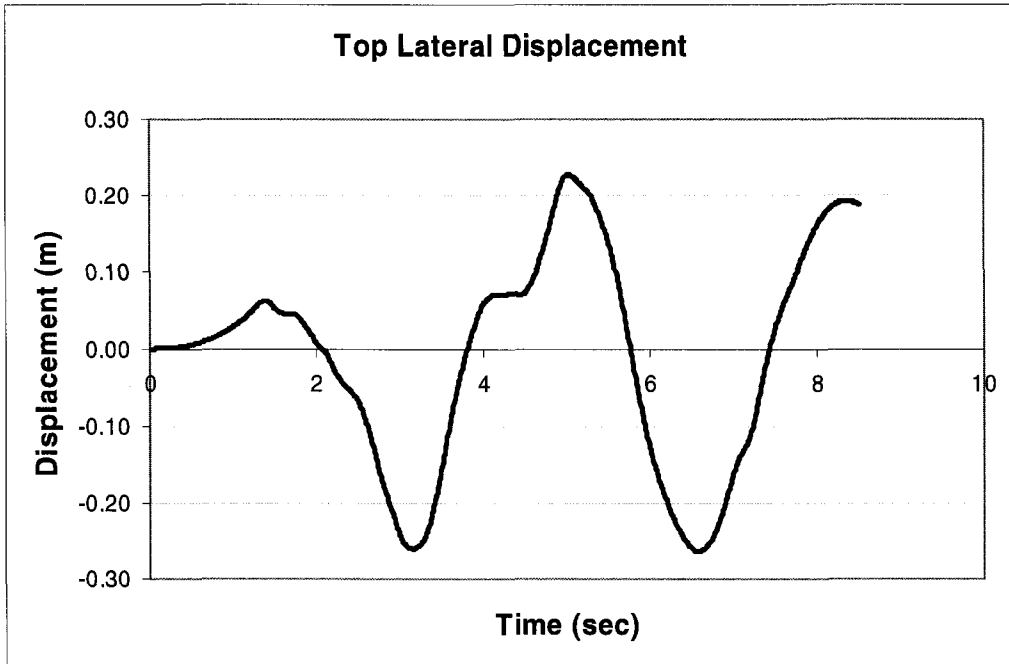


Figure 4.38 Top lateral displacement response of the 15 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

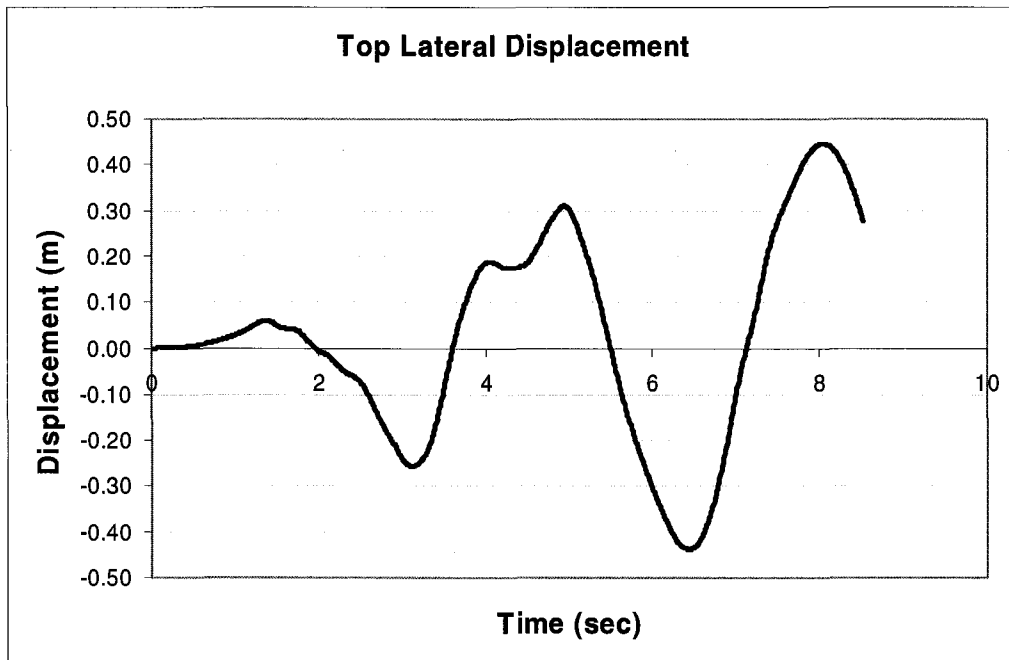


Figure 4.39 Top lateral displacement response of the 15 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

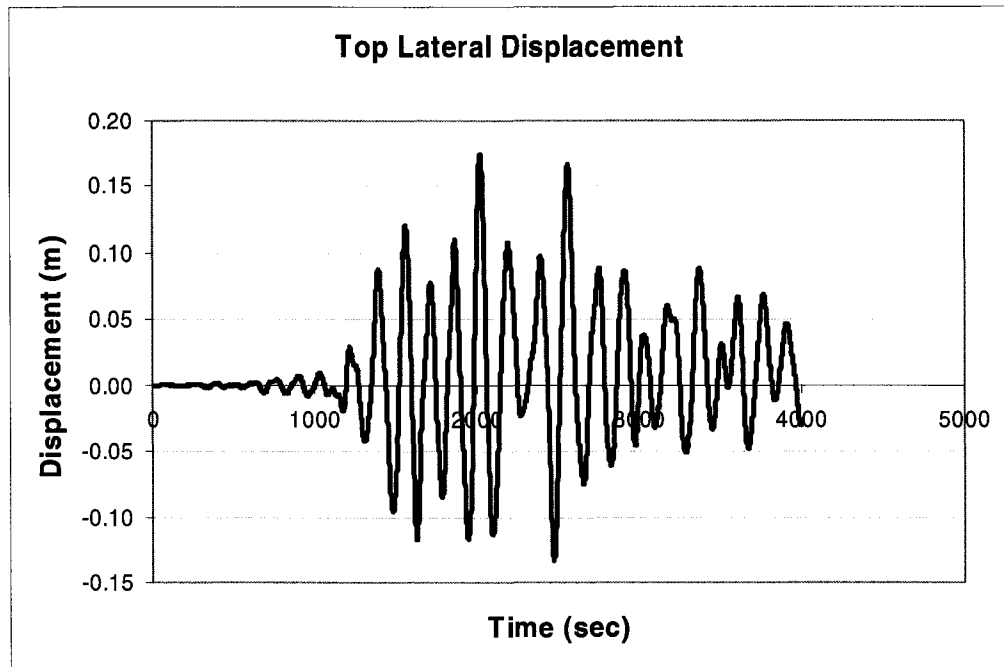


Figure 4.40 Top lateral displacement response of the 5 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Nisqually (2001)'

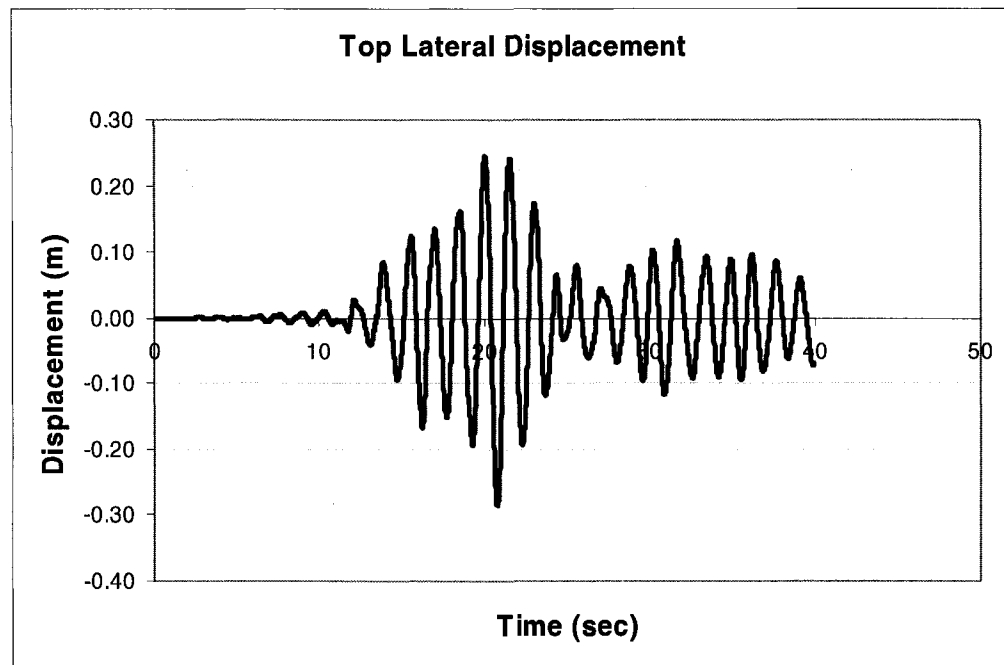


Figure 4.41 Top lateral displacement response of the 5 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Nisqually (2001)'

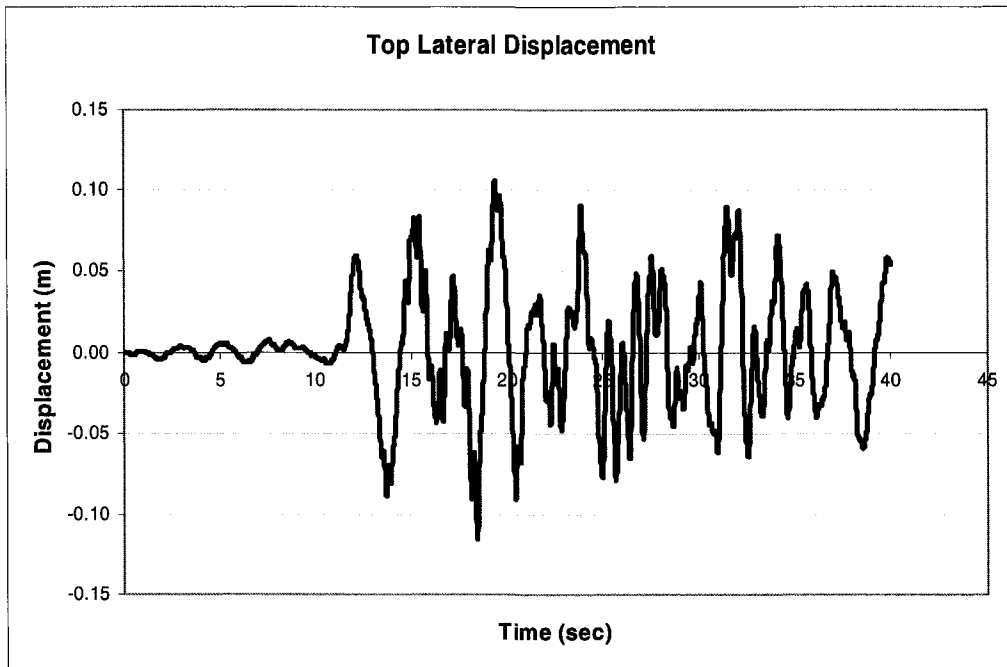


Figure 4.42 Top lateral displacement response of the 10 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

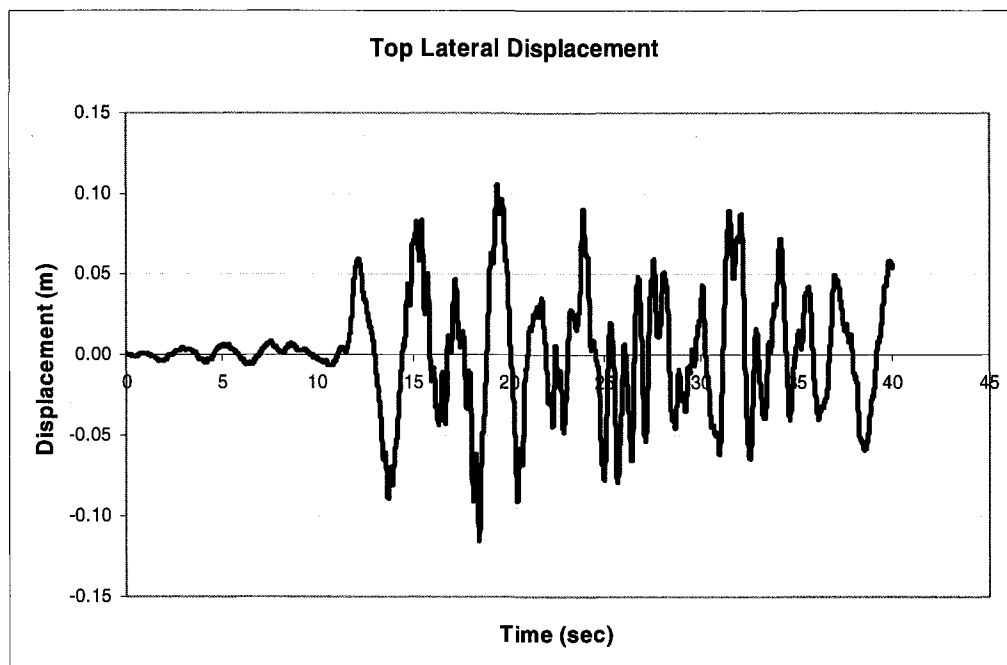


Figure 4.43 Top lateral displacement response of the 10 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

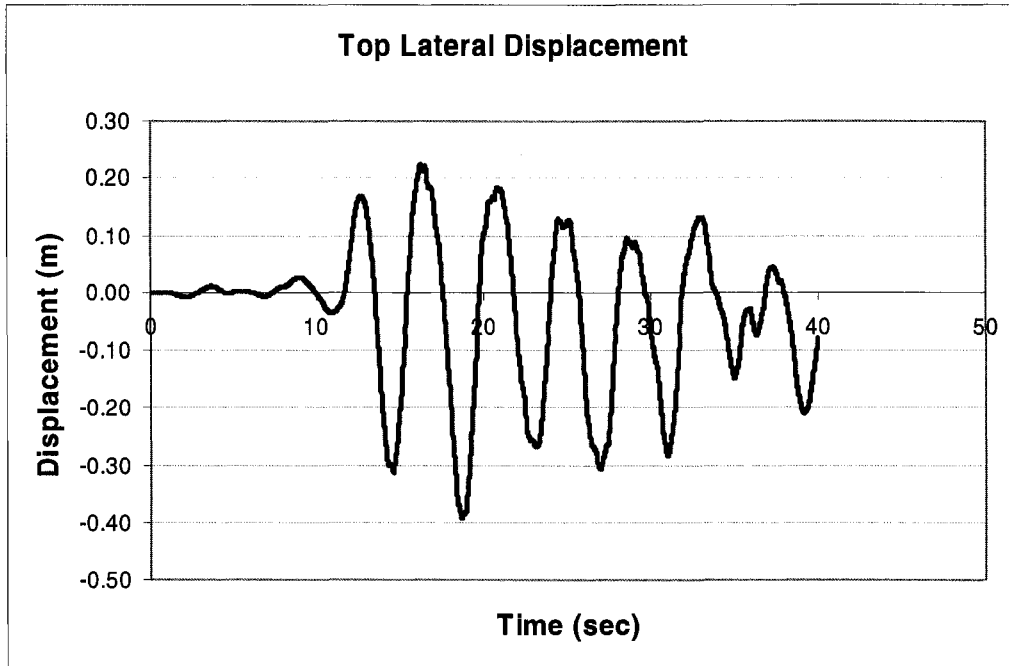


Figure 4.44 top Lateral displacement response of the 15 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

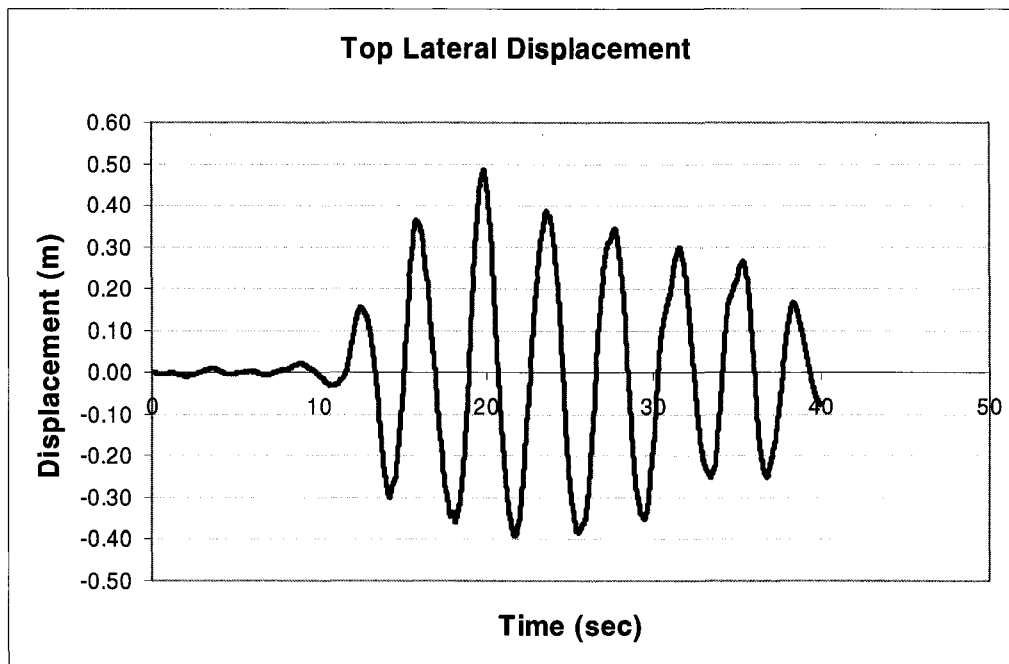


Figure 4.45 Top lateral displacement response of the 15 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

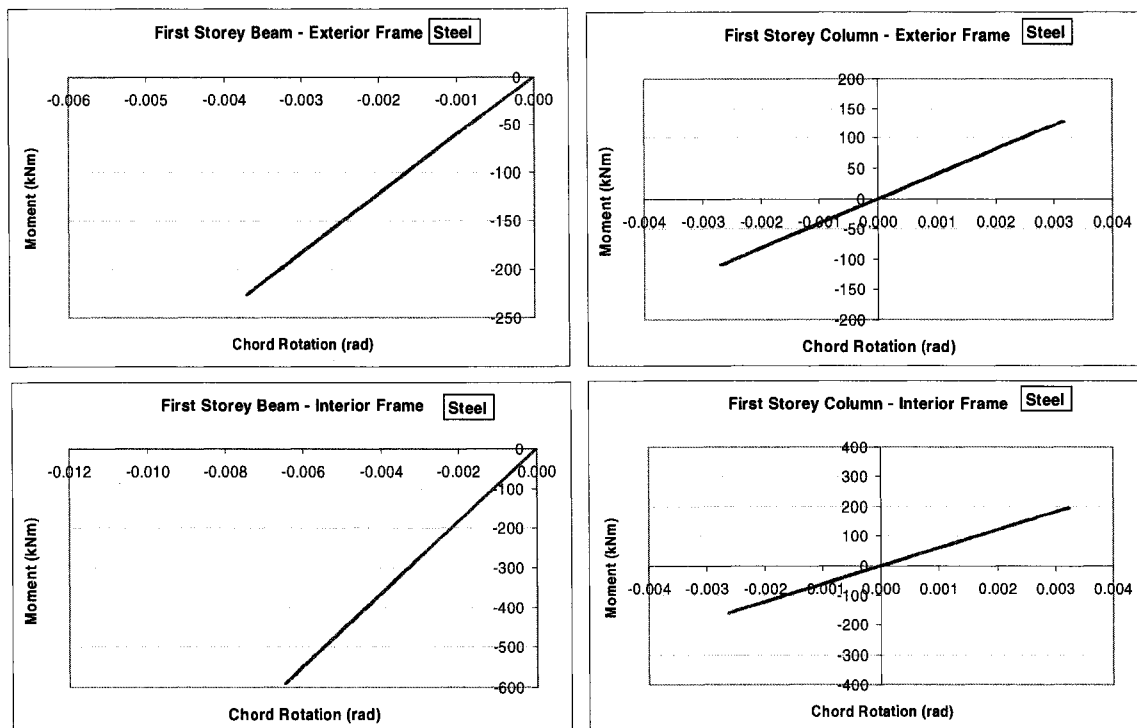


Figure 4.46 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

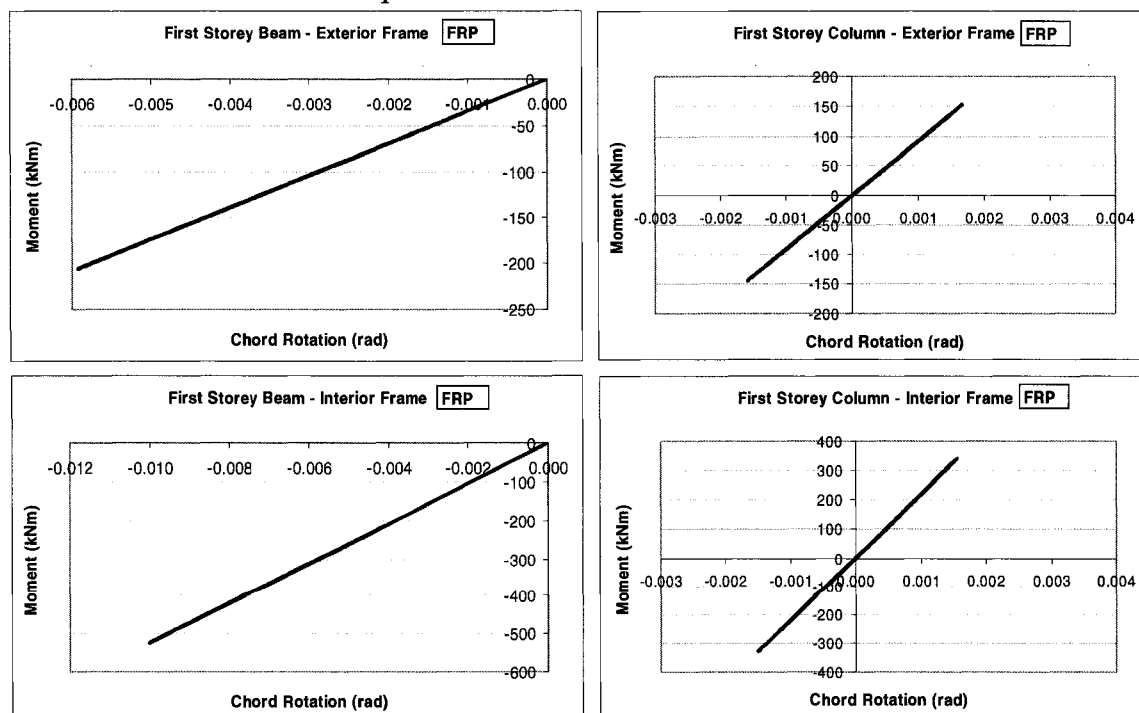


Figure 4.47 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

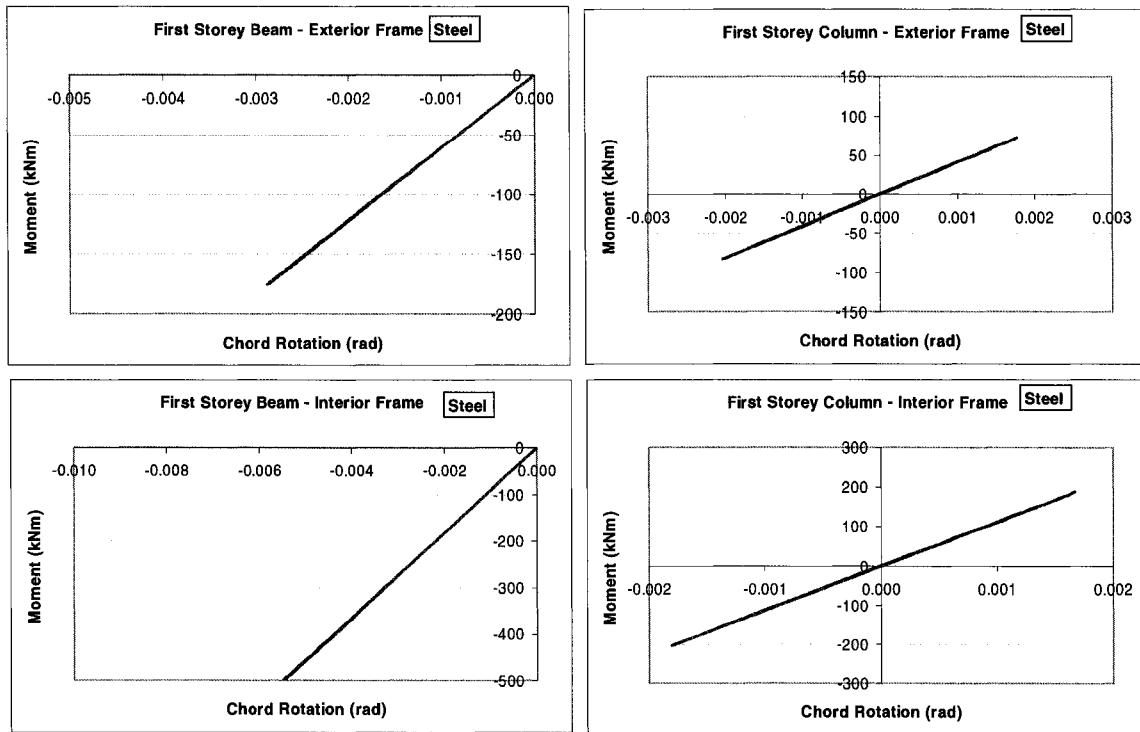


Figure 4.48 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

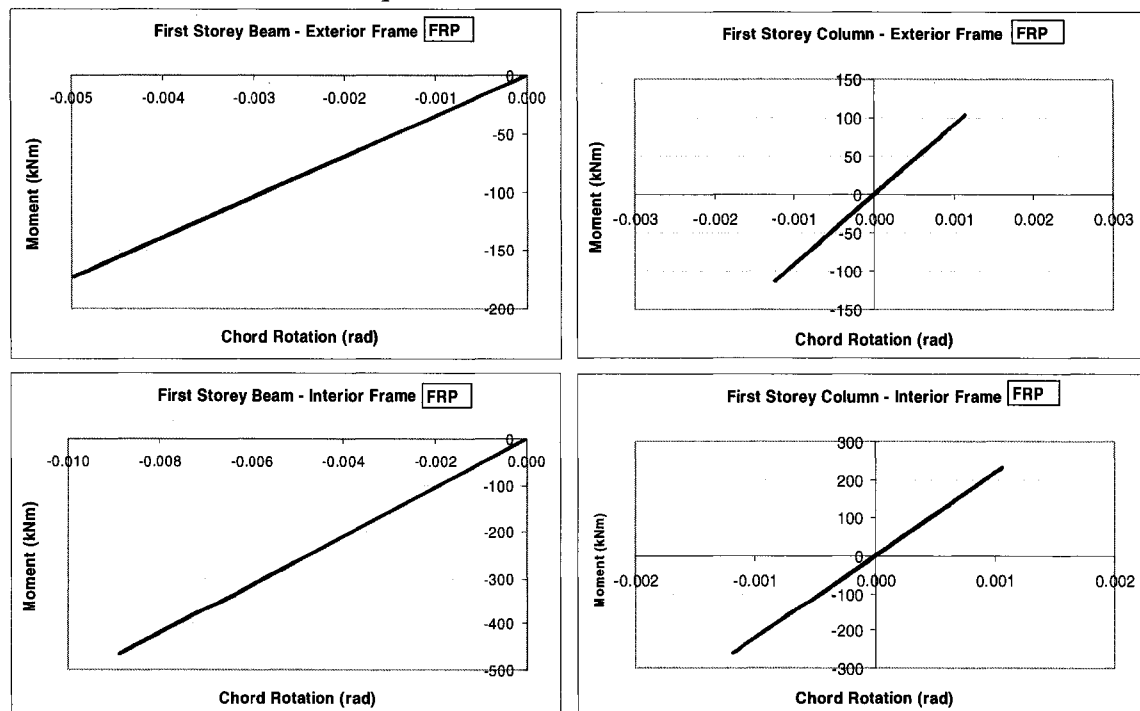


Figure 4.49 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

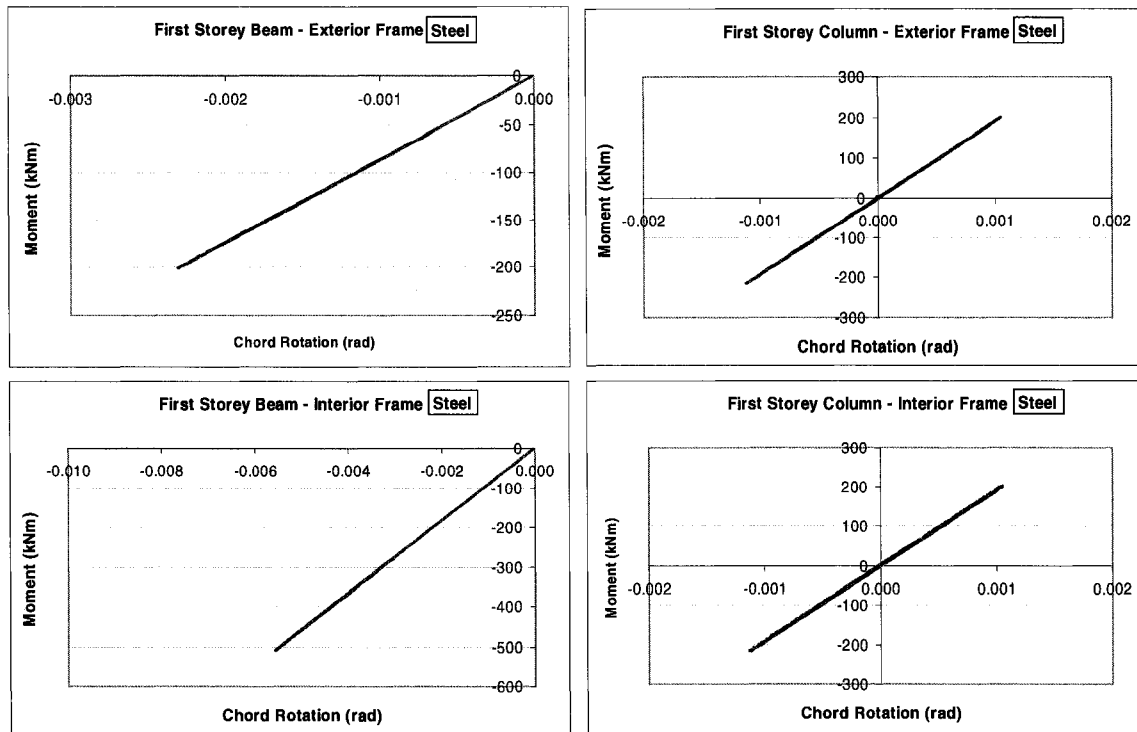


Figure 4.50 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey steel reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

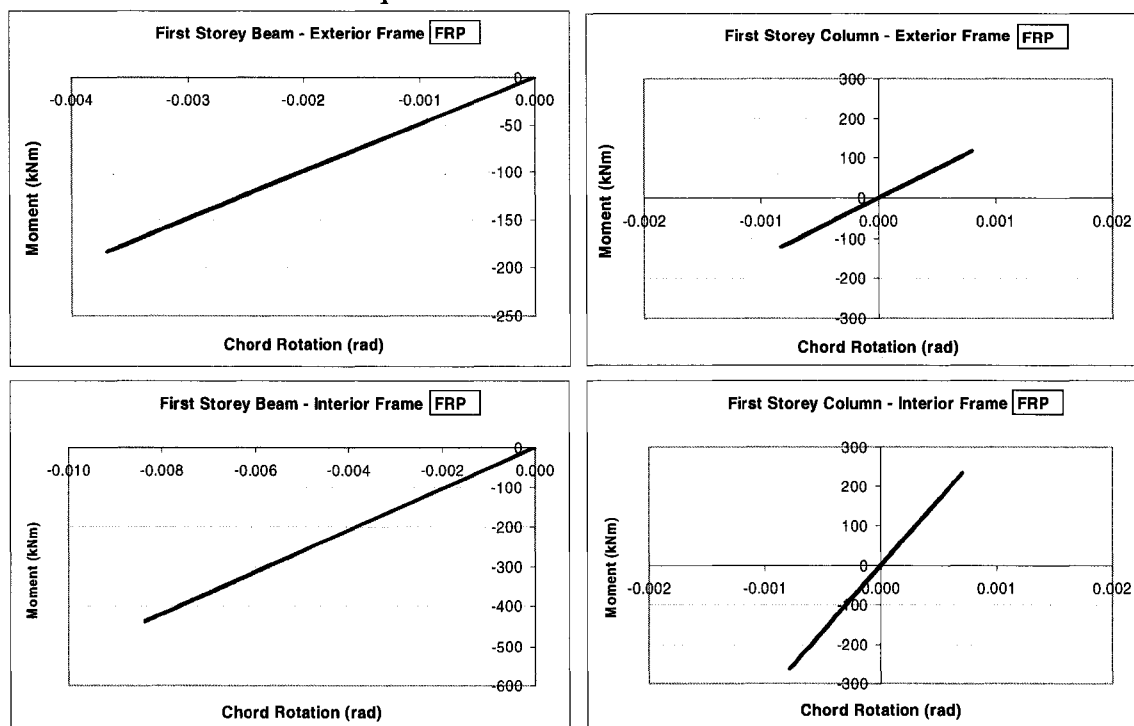


Figure 4.51 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey FRP reinforced building in Ottawa, analyzed under the artificially generated earthquake record 'Short Event 4-Ottawa'

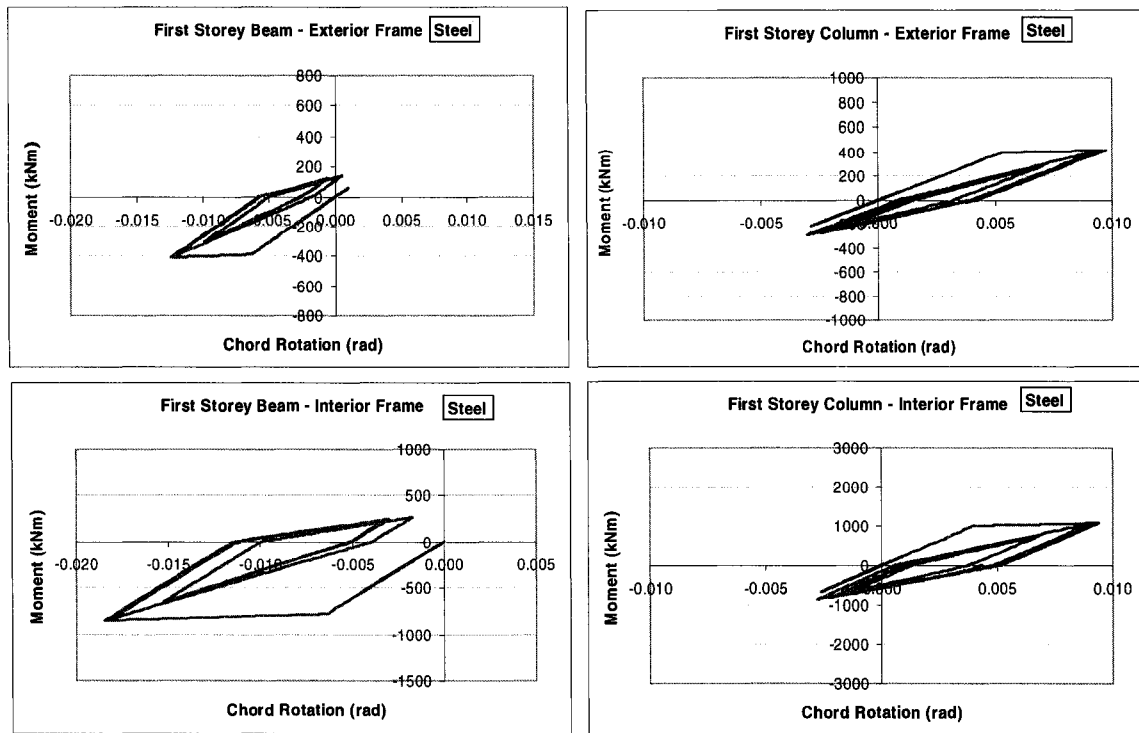


Figure 4.52 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

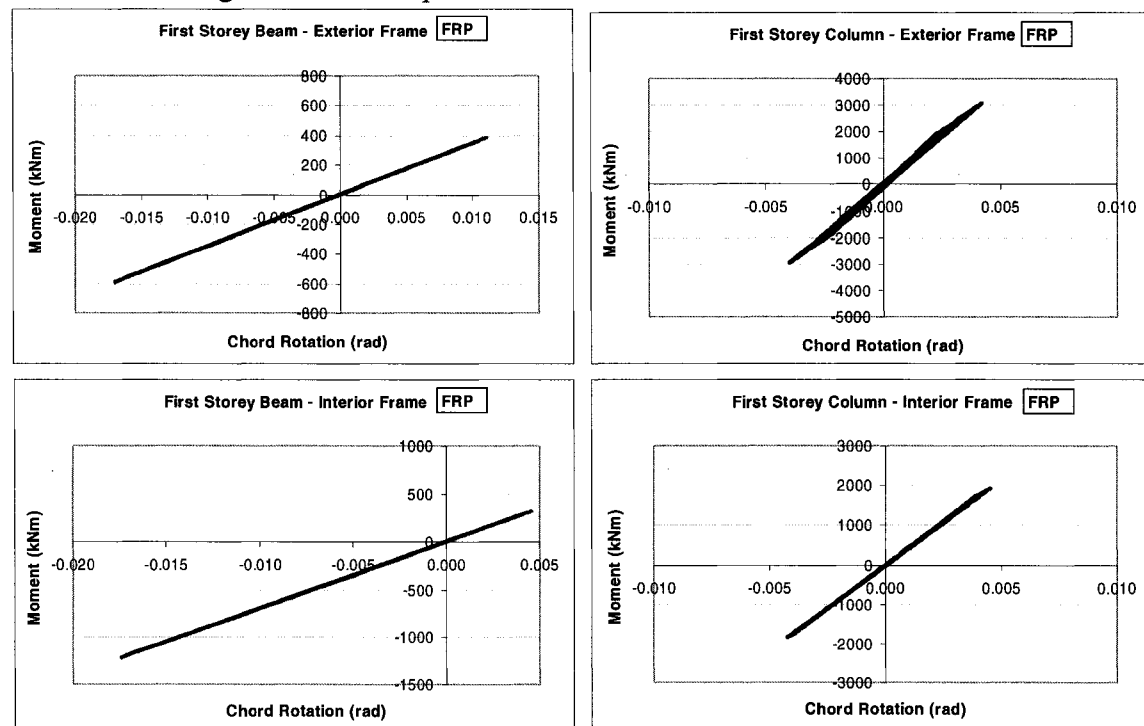


Figure 4.53 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

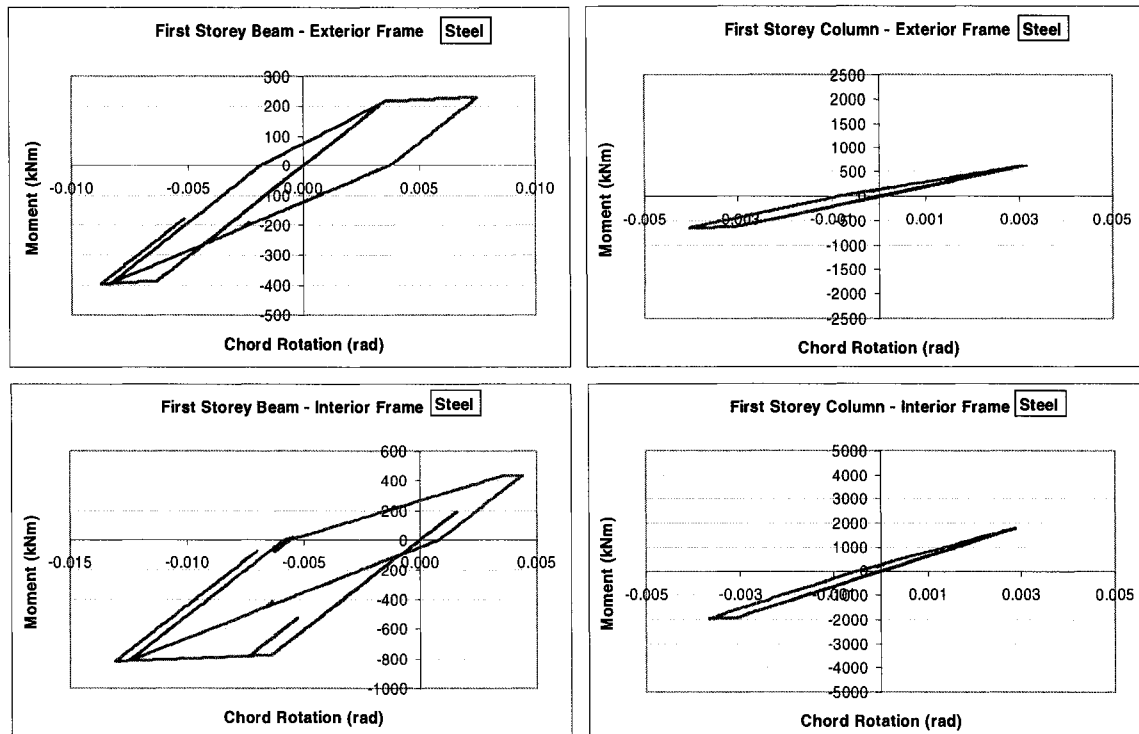


Figure 4.54 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

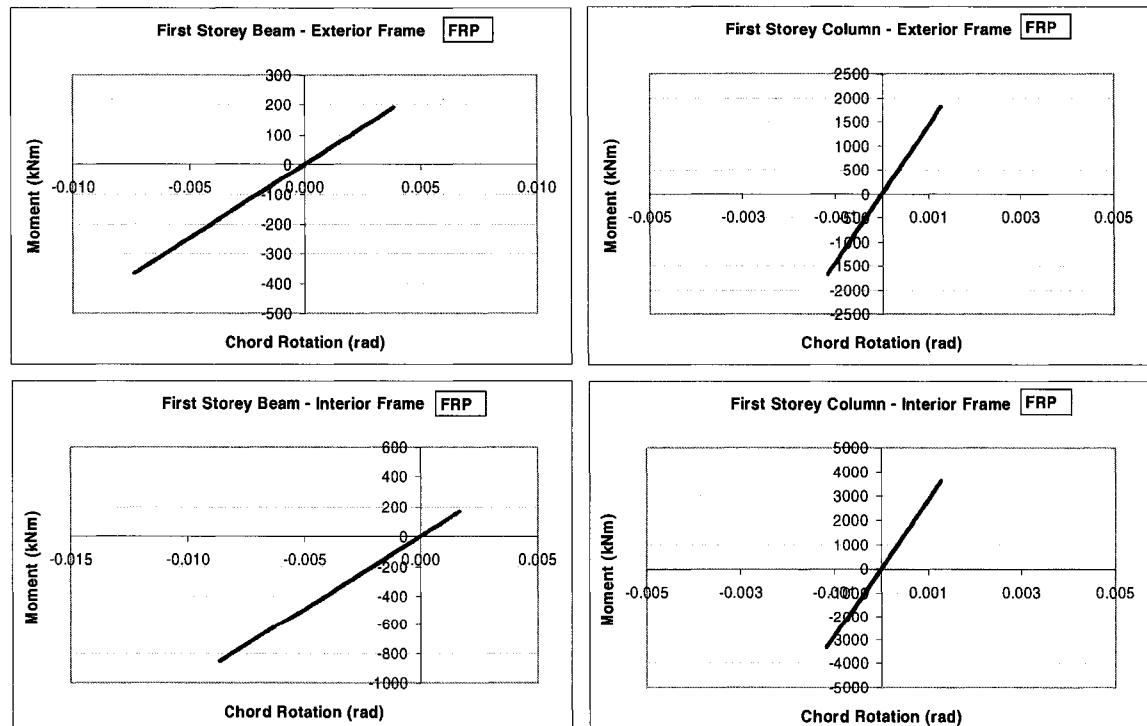


Figure 4.55 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

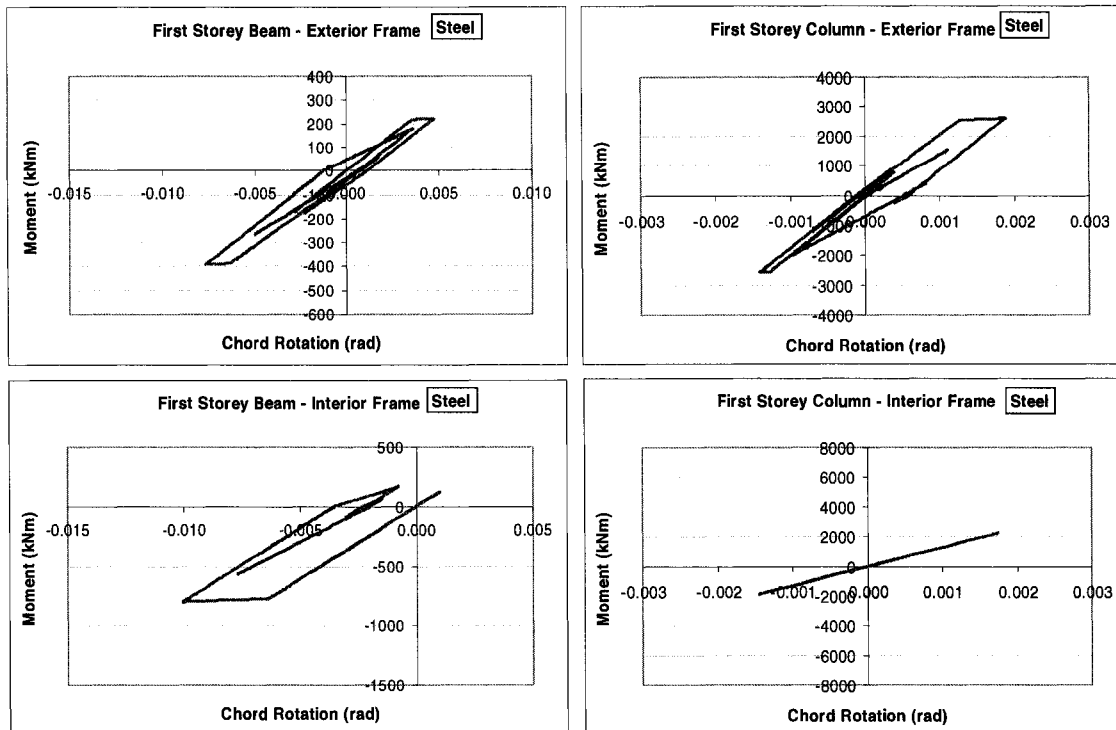


Figure 4.56 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey steel reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

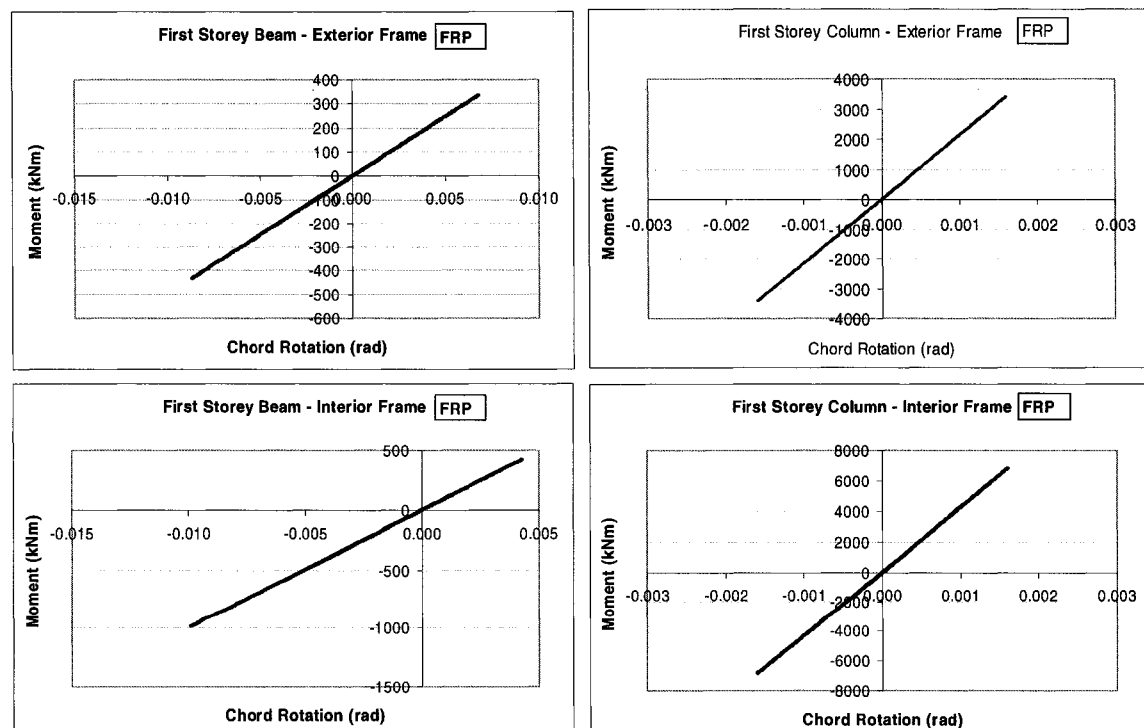


Figure 4.57 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey FRP reinforced building in Vancouver, analyzed under the artificially generated earthquake record 'Short Event 4- Vancouver'

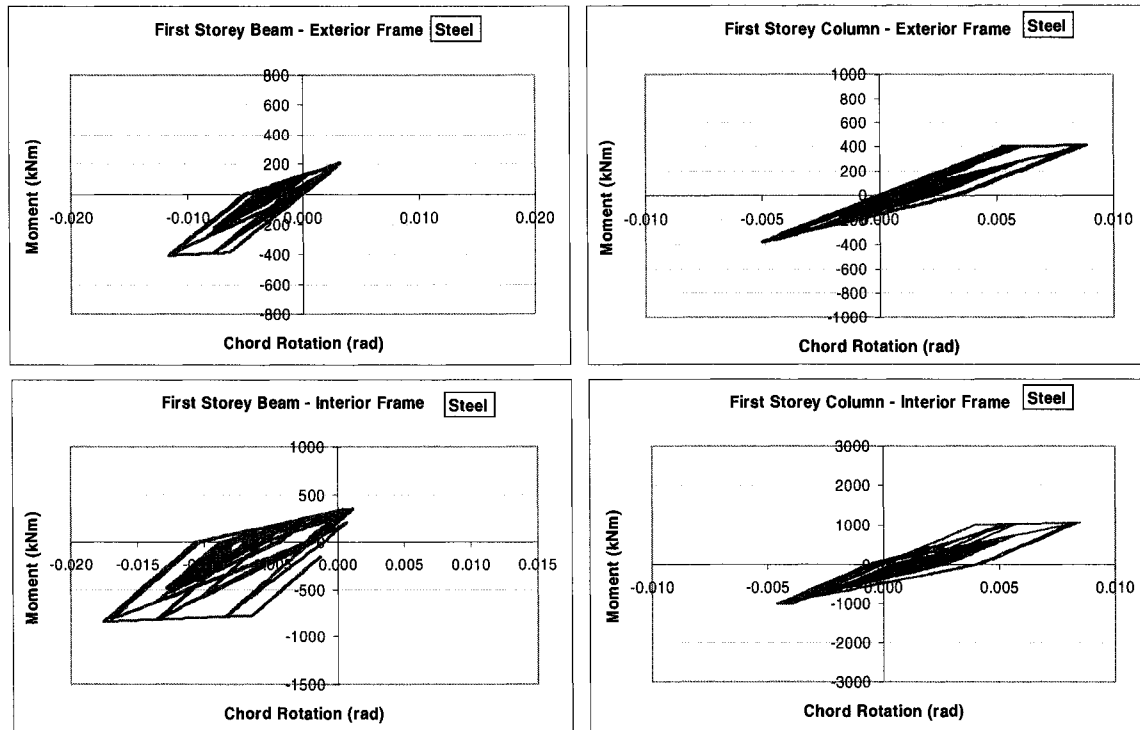


Figure 4.58 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Nisqually (2001)'

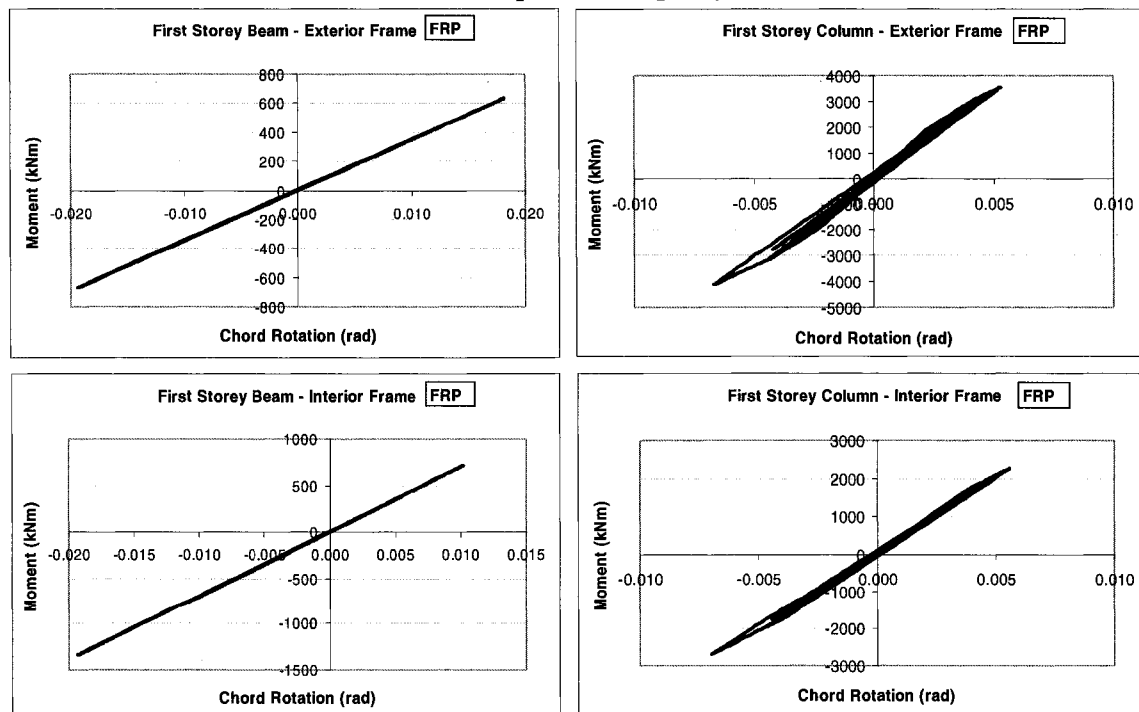


Figure 4.59 Moment-chord rotation hysteretic relationship for the beams and columns of the 5 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Nisqually (2001)'

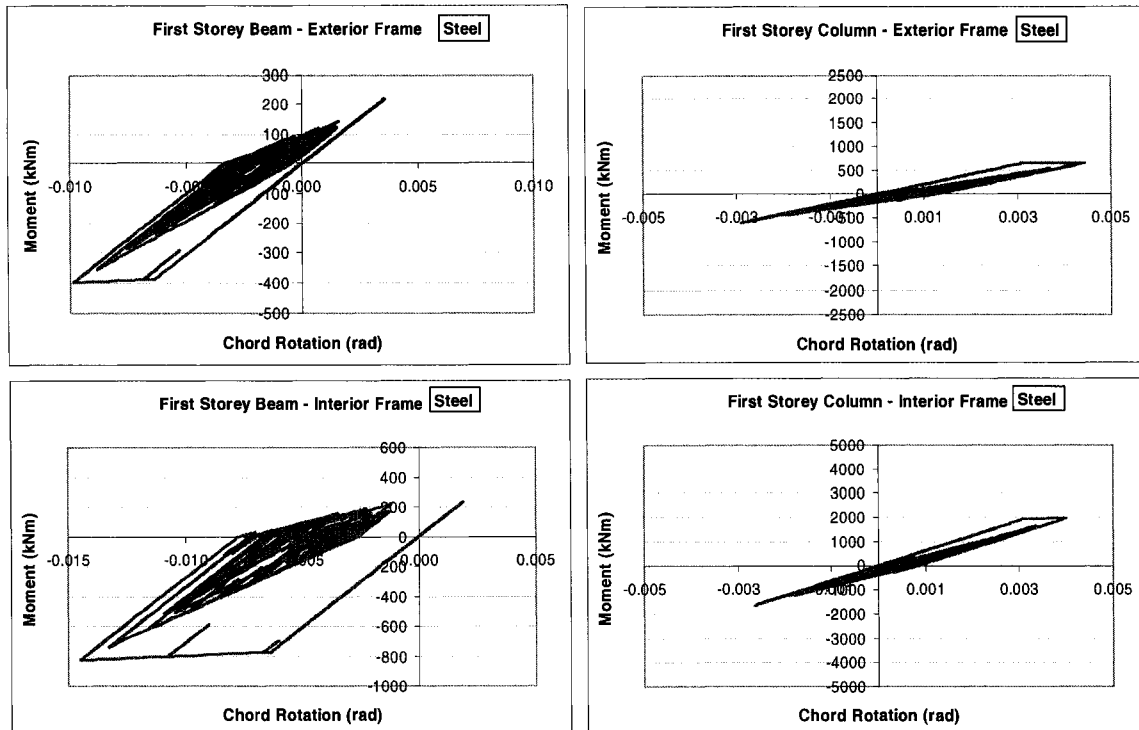


Figure 4.60 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

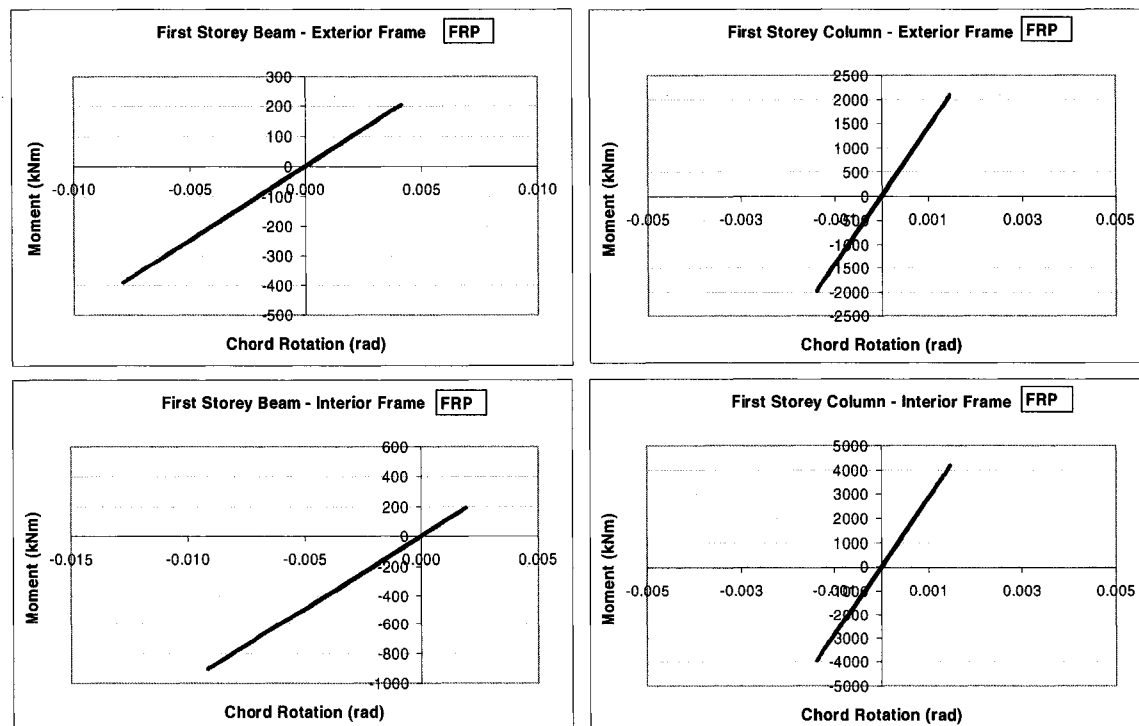


Figure 4.61 Moment-chord rotation hysteretic relationship for the beams and columns of the 10 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

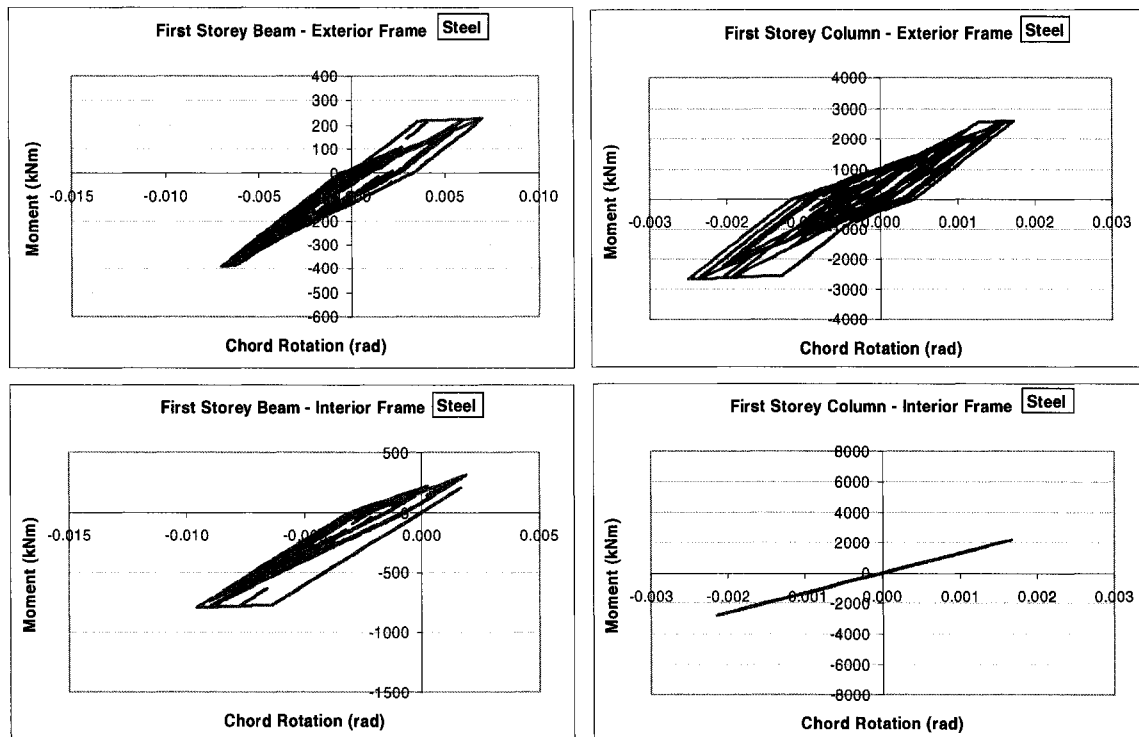


Figure 4.62 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey steel reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

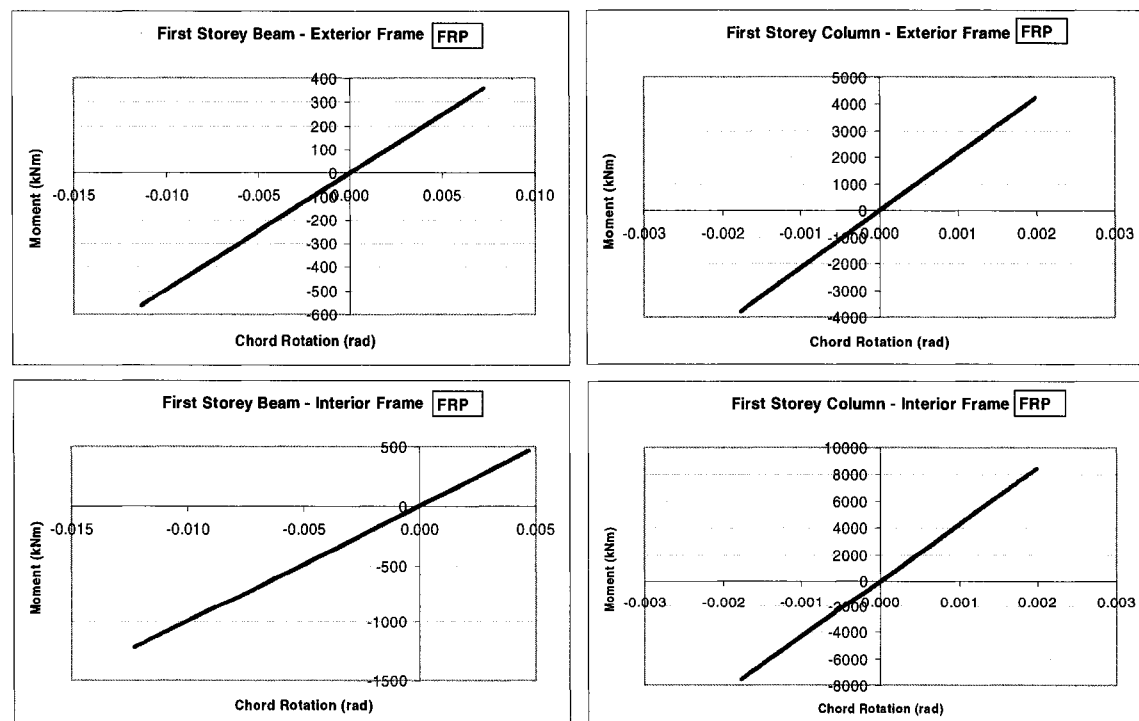


Figure 4.63 Moment-chord rotation hysteretic relationship for the beams and columns of the 15 storey FRP reinforced building in Vancouver, analyzed under the previously recorded earthquake 'Tokachi Oki (2003)'

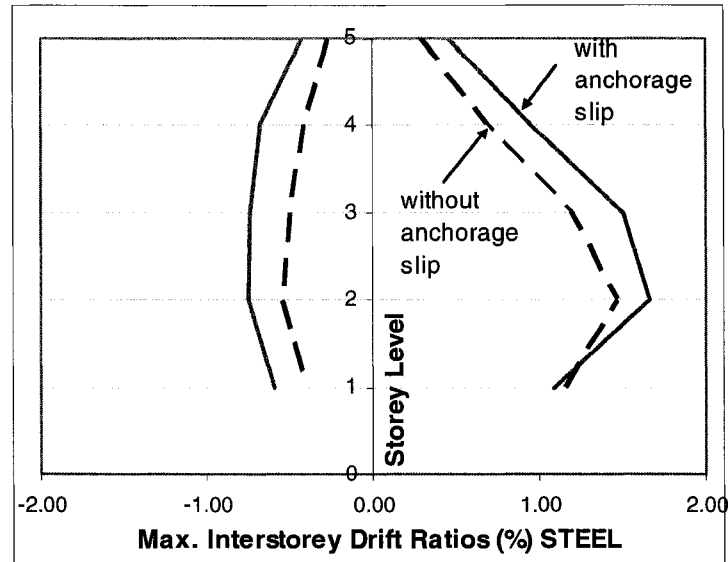


Figure 4.64 Maximum interstorey drift ratios for the 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

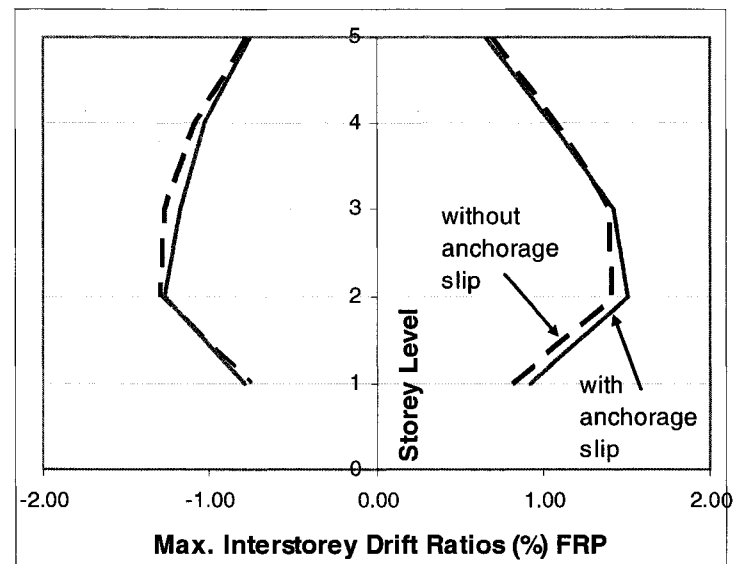


Figure 4.65 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

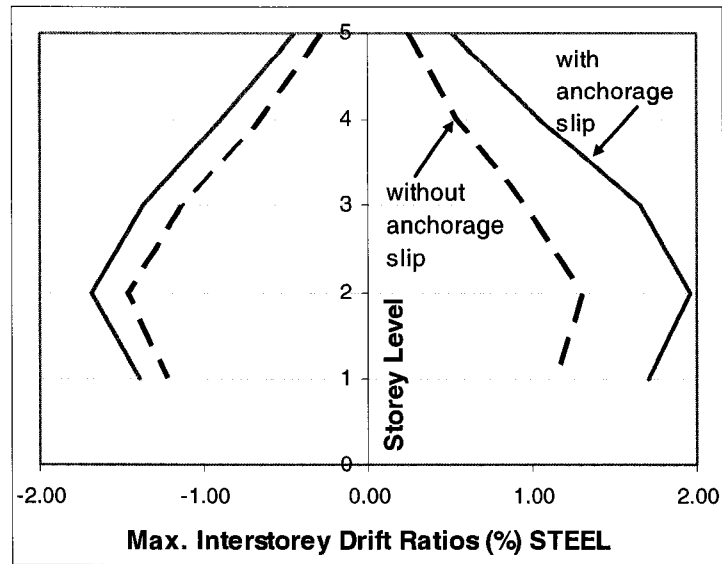


Figure 4.66 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Long Event 1-Vancouver'

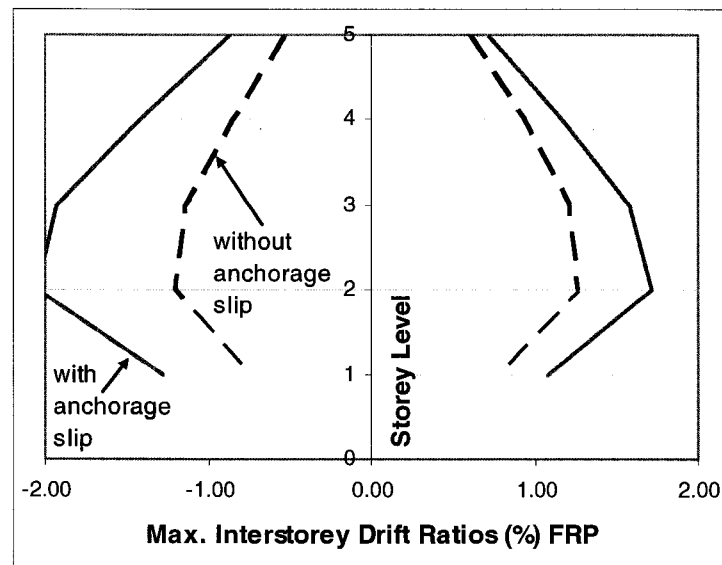


Figure 4.67 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Long Event 1-Vancouver'

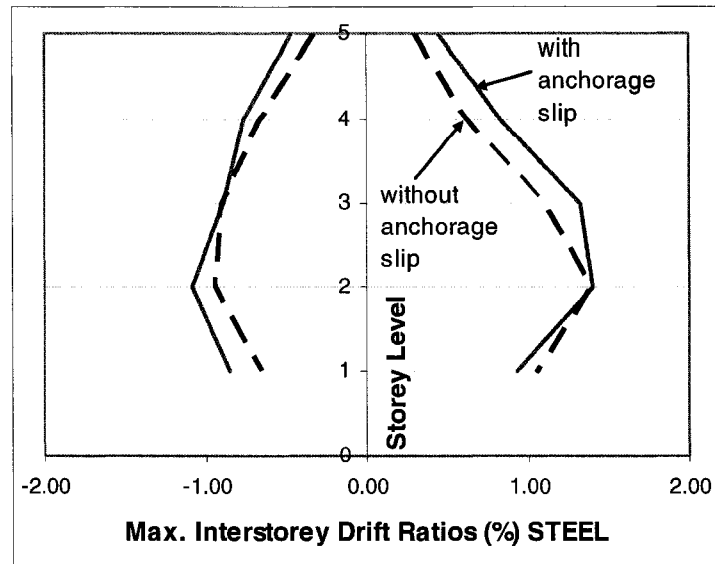


Figure 4.68 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the previously recorded earthquake 'Nisqually (2001)'

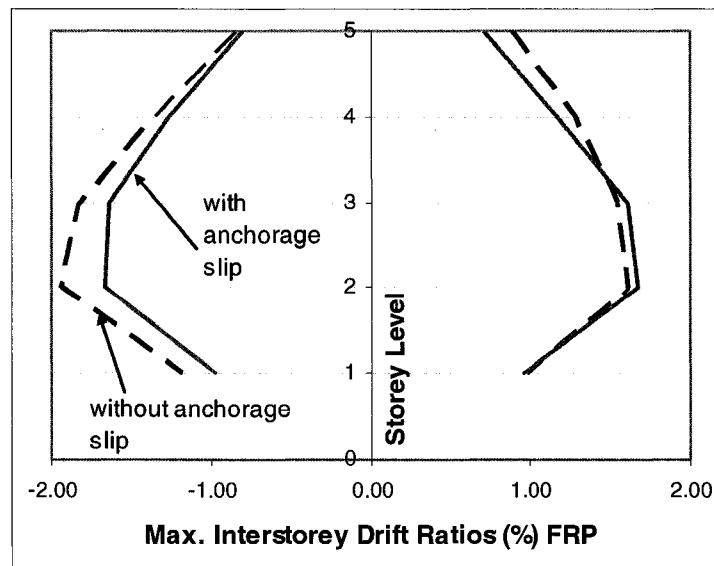


Figure 4.69 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect analyzed under the previously recorded earthquake 'Nisqually (2001)'

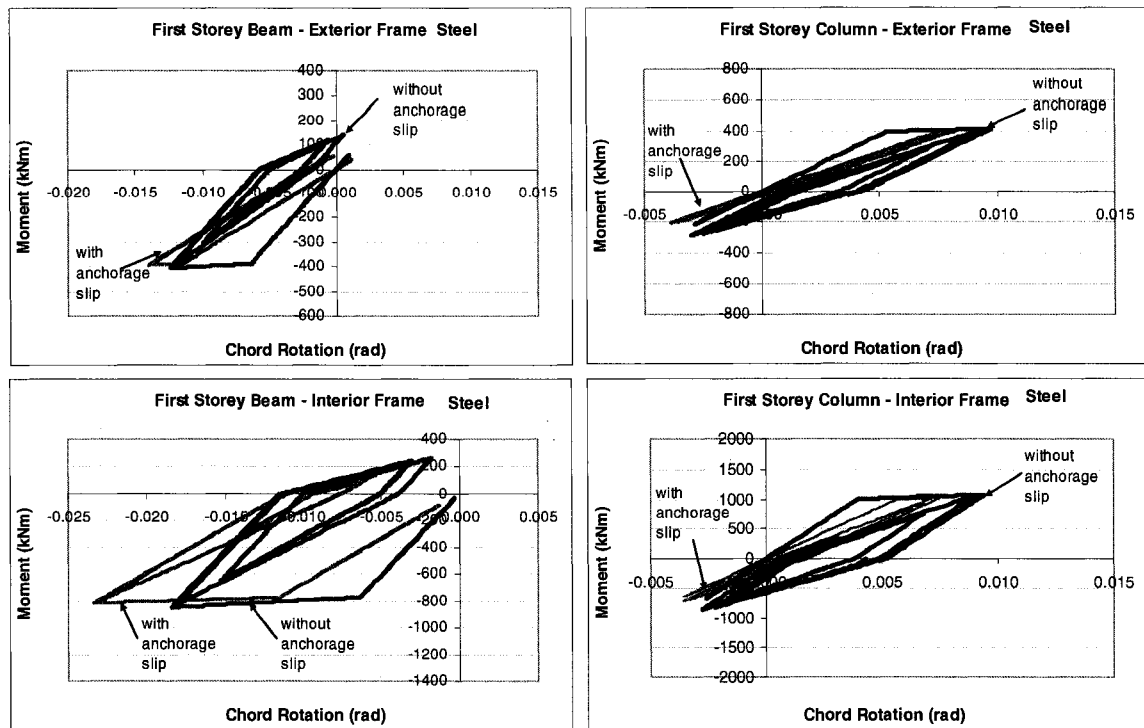


Figure 4.70 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

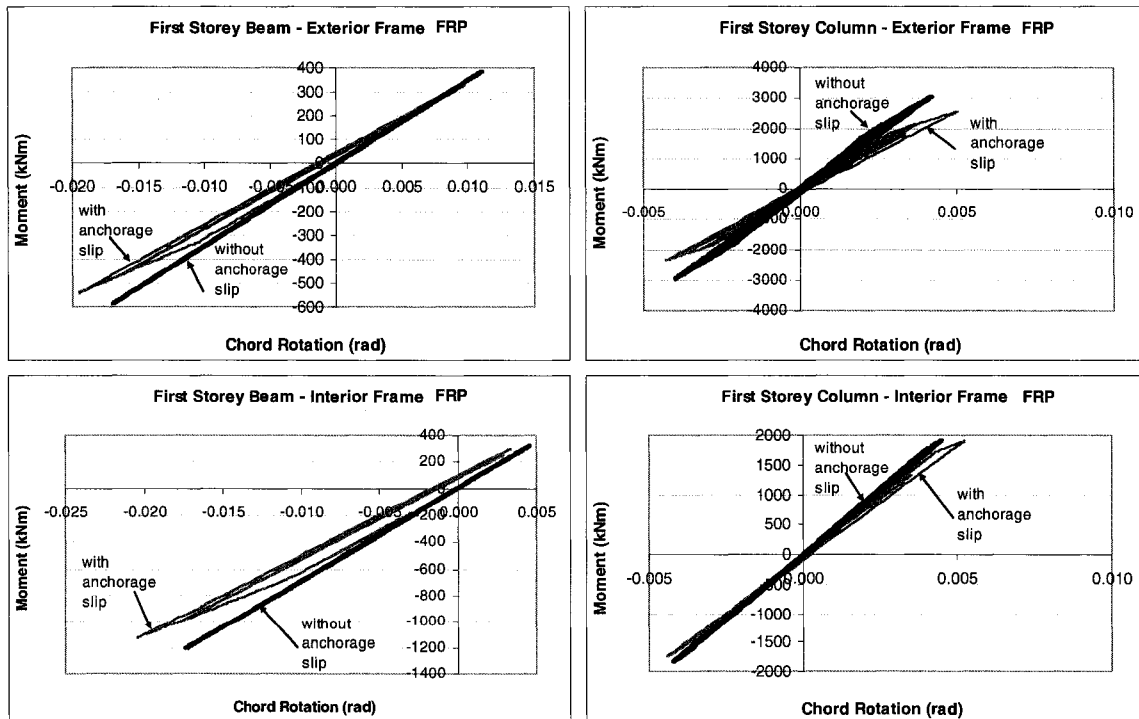


Figure 4.71 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Short Event 4-Vancouver'

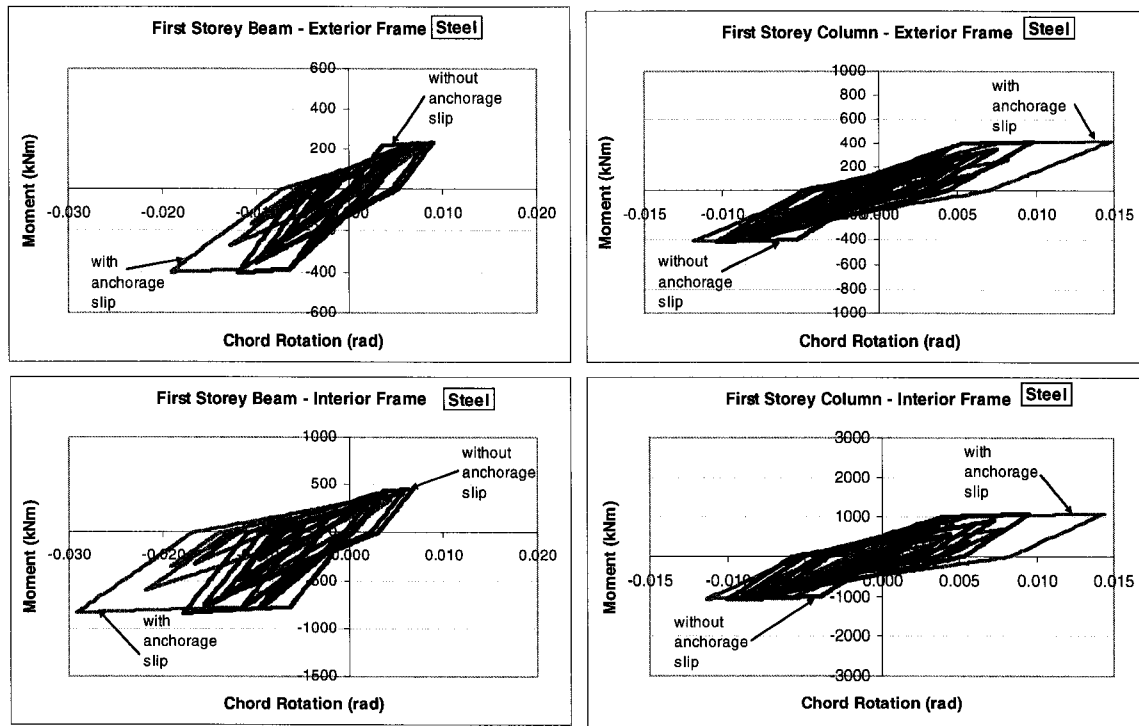


Figure 4.72 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Long Event 1-Vancouver'

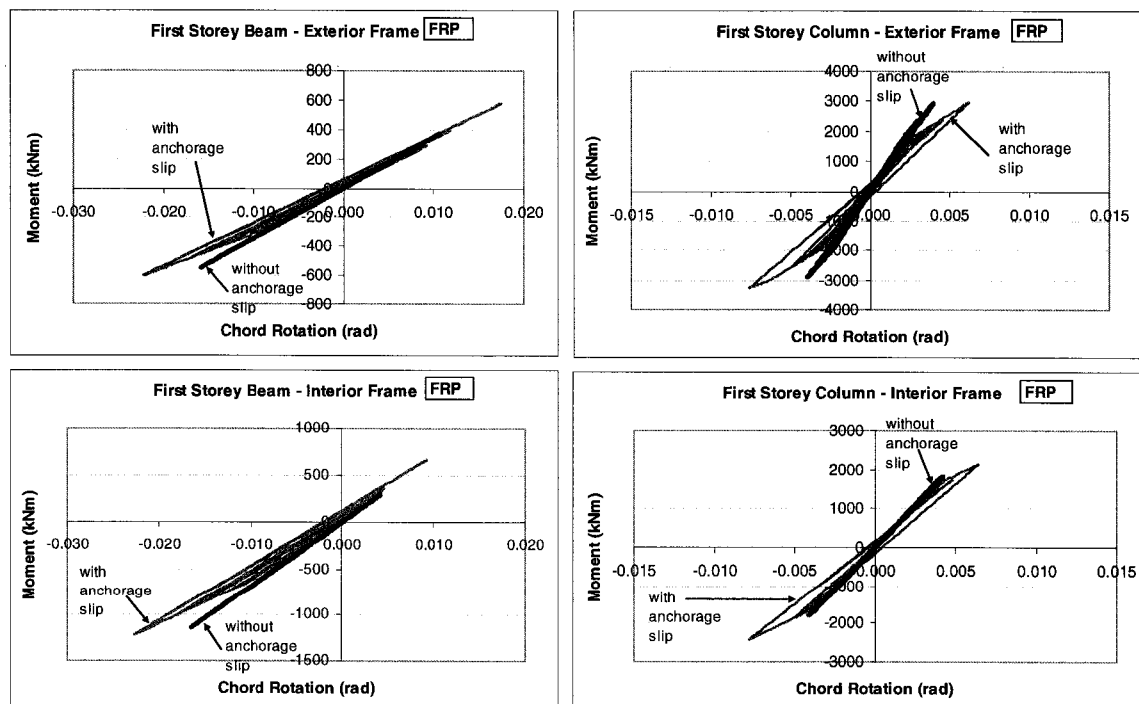


Figure 4.73 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the artificially generated earthquake record 'Long Event 1-Vancouver'

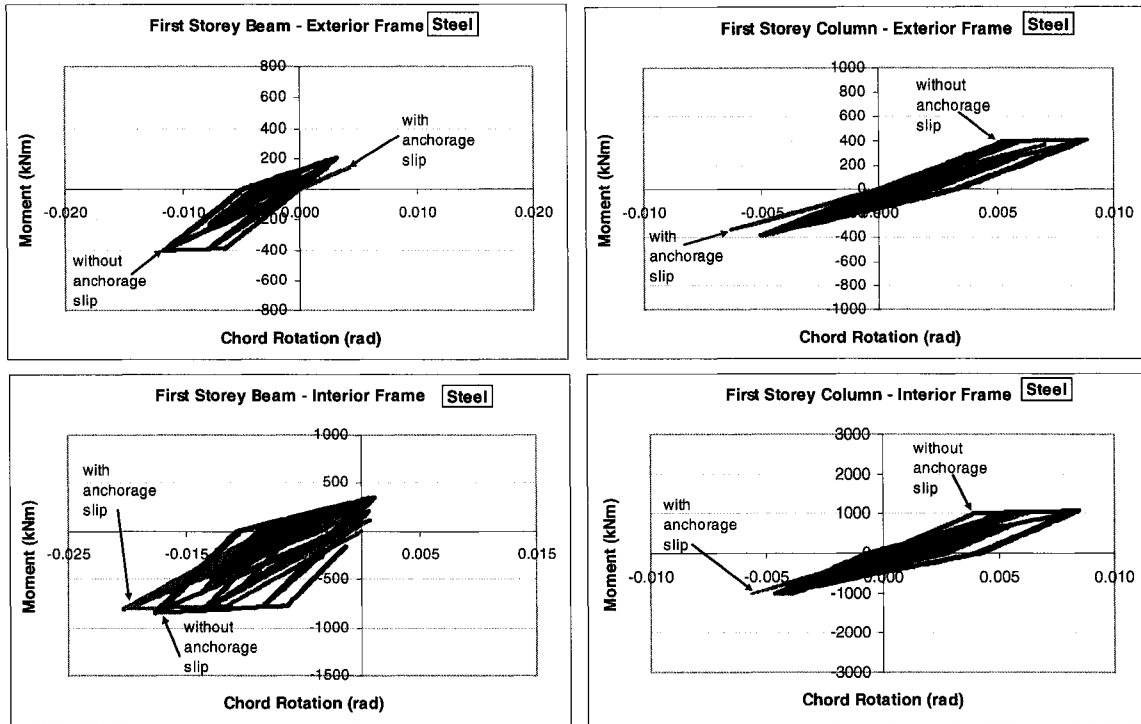


Figure 4.74 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the previously recorded earthquake 'Nisqually (2001)'

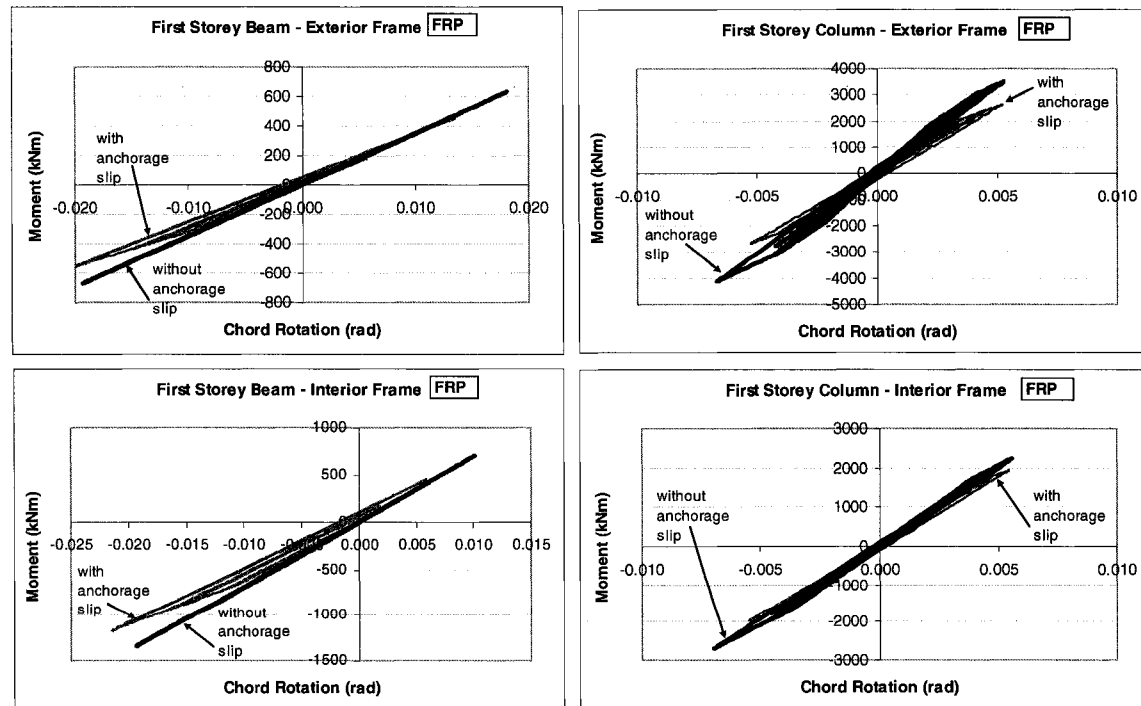


Figure 4.75 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver, with and without anchorage slip effect, analyzed under the previously recorded earthquake 'Nisqually (2001)'

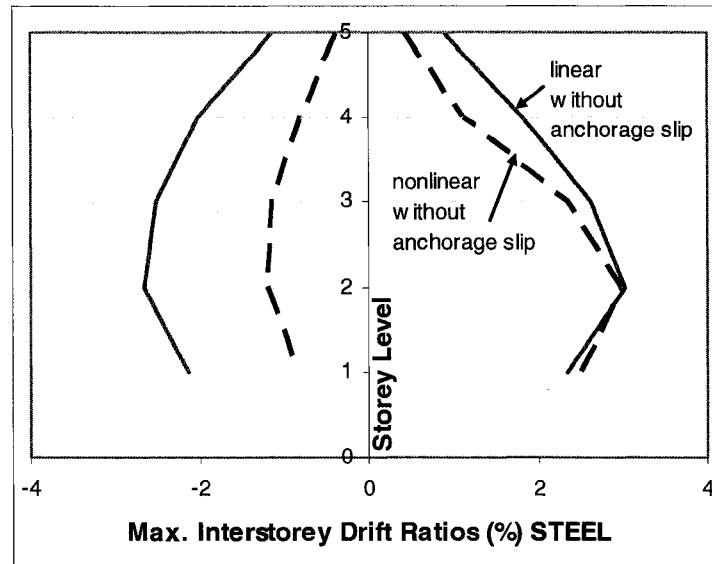


Figure 4.76 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Short Event 4-Vancouver'

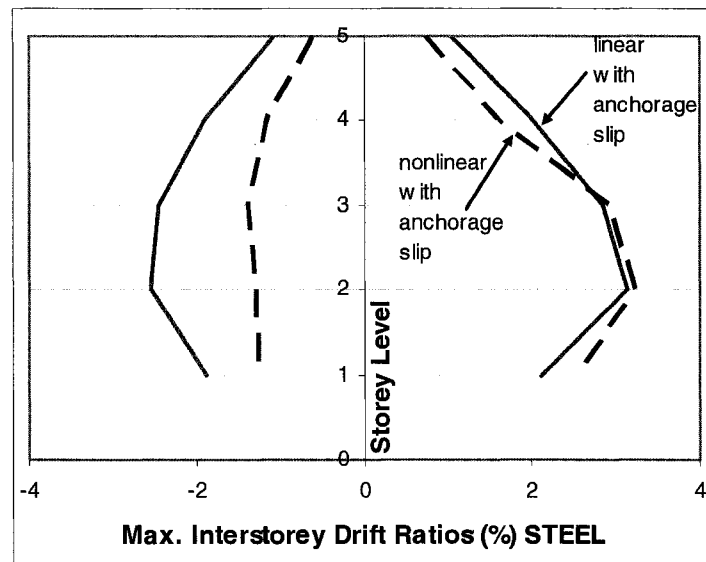


Figure 4.77 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, with anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Short Event 4-Vancouver'

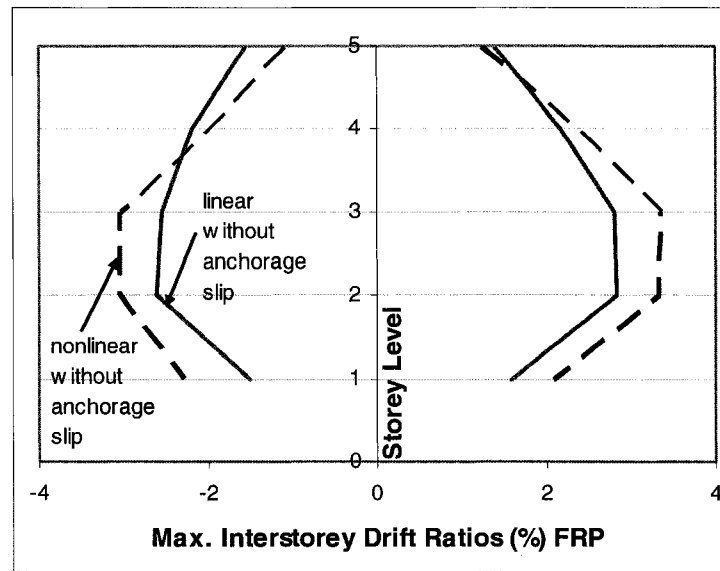


Figure 4.78 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Short Event 4-Vancouver'

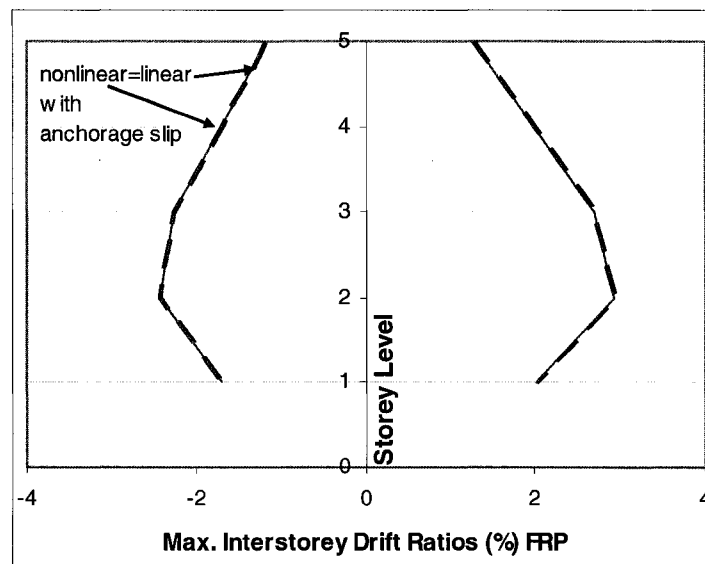


Figure 4.79 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, with anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Short Event 4-Vancouver'

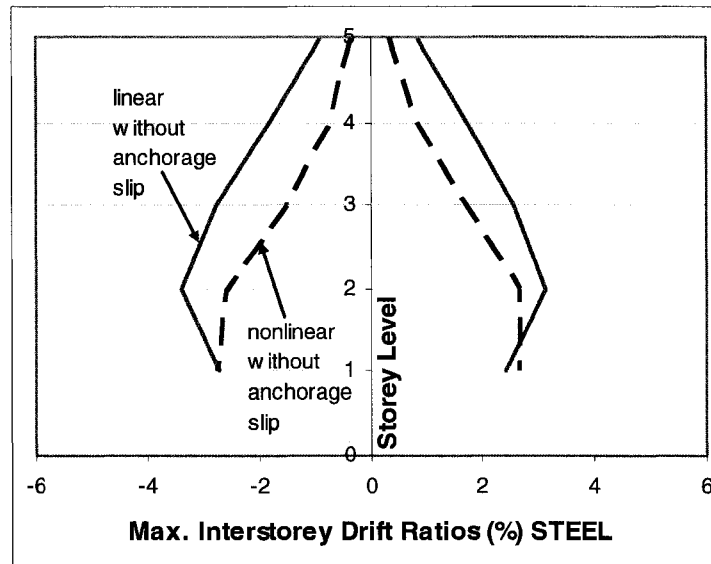


Figure 4.80 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Long Event 1-Vancouver'

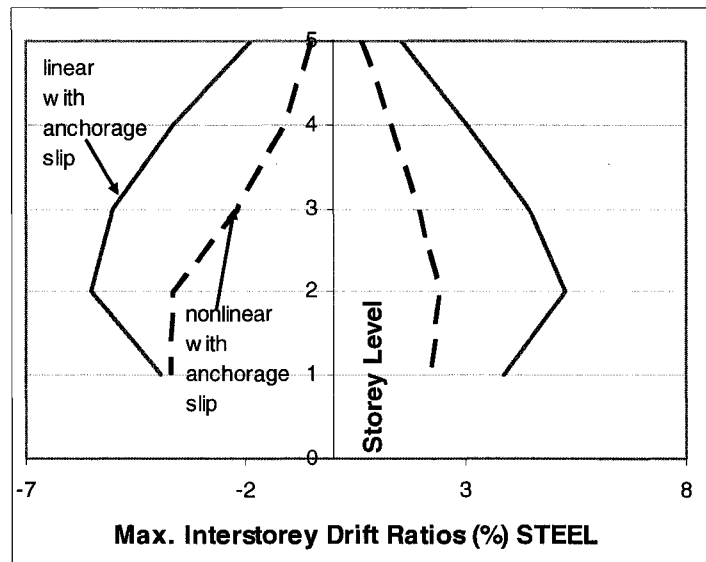


Figure 4.81 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, with anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Long Event 1-Vancouver'

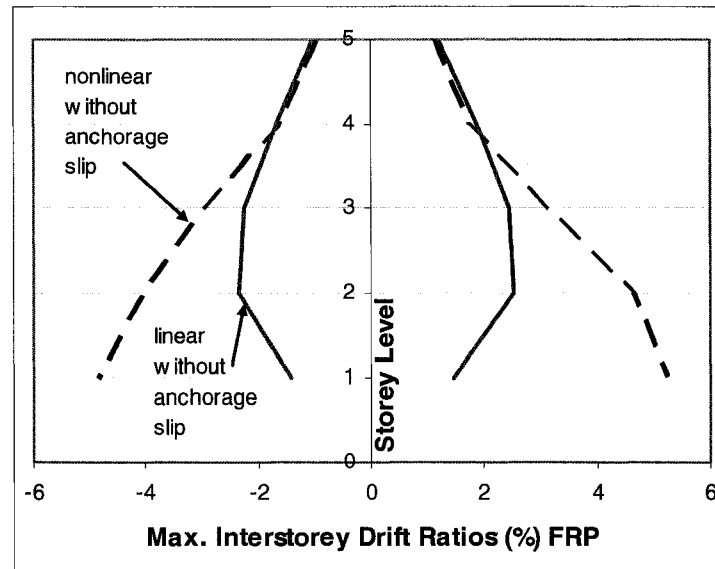


Figure 4.82 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Long Event 1-Vancouver'

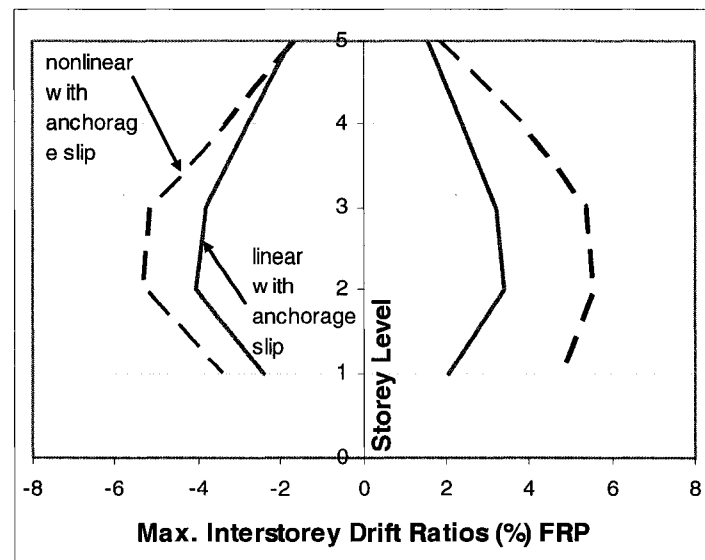


Figure 4.83 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, with anchorage slip effect, analyzed under the 100% intensified artificially generated earthquake record 'Long Event 1-Vancouver'

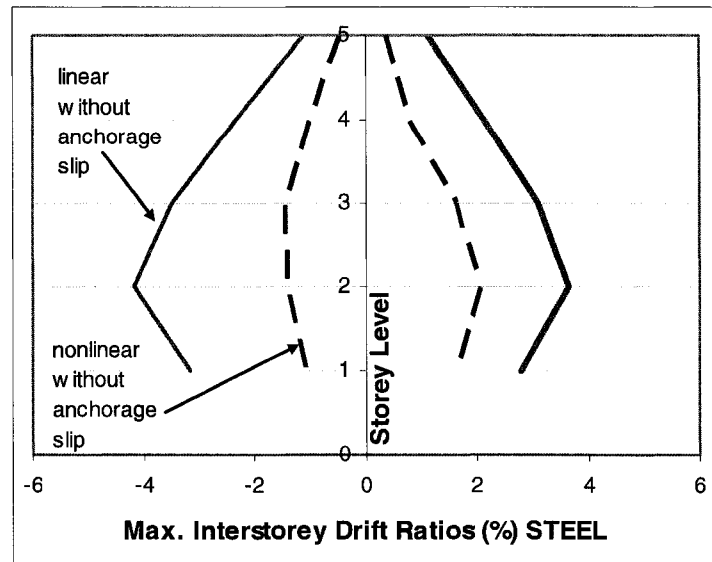


Figure 4.84 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified previously recorded earthquake 'Nisqually (2001)'

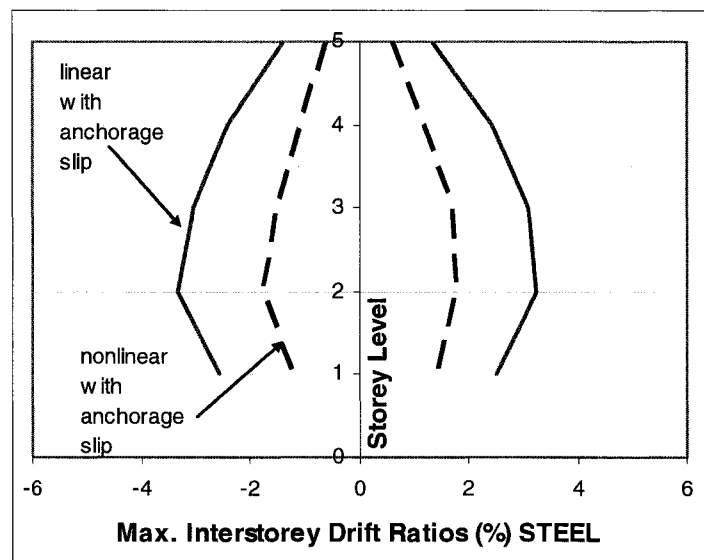


Figure 4.85 Maximum interstorey drift ratios of 5 storey steel reinforced building in Vancouver, with anchorage slip effect, analyzed under the 100% intensified previously recorded earthquake 'Nisqually (2001)'

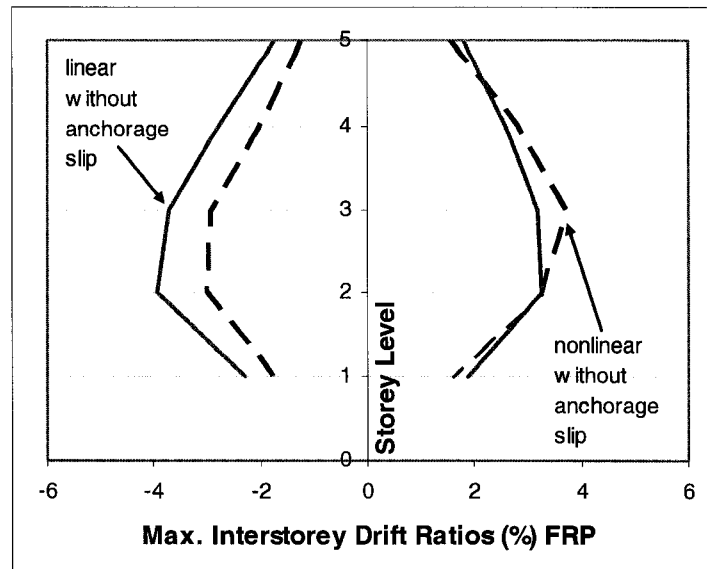


Figure 4.86 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified previously recorded earthquake 'Nisqually (2001)'

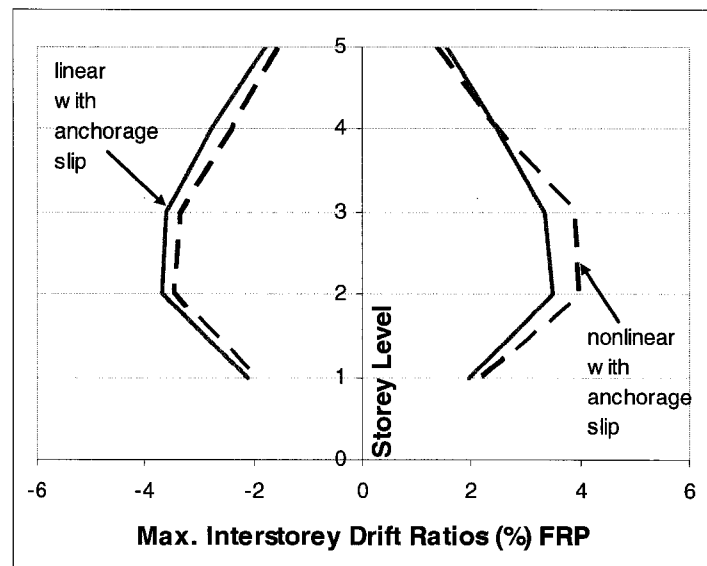


Figure 4.87 Maximum interstorey drift ratios of 5 storey FRP reinforced building in Vancouver, without anchorage slip effect, analyzed under the 100% intensified previously recorded earthquake 'Nisqually (2001)'

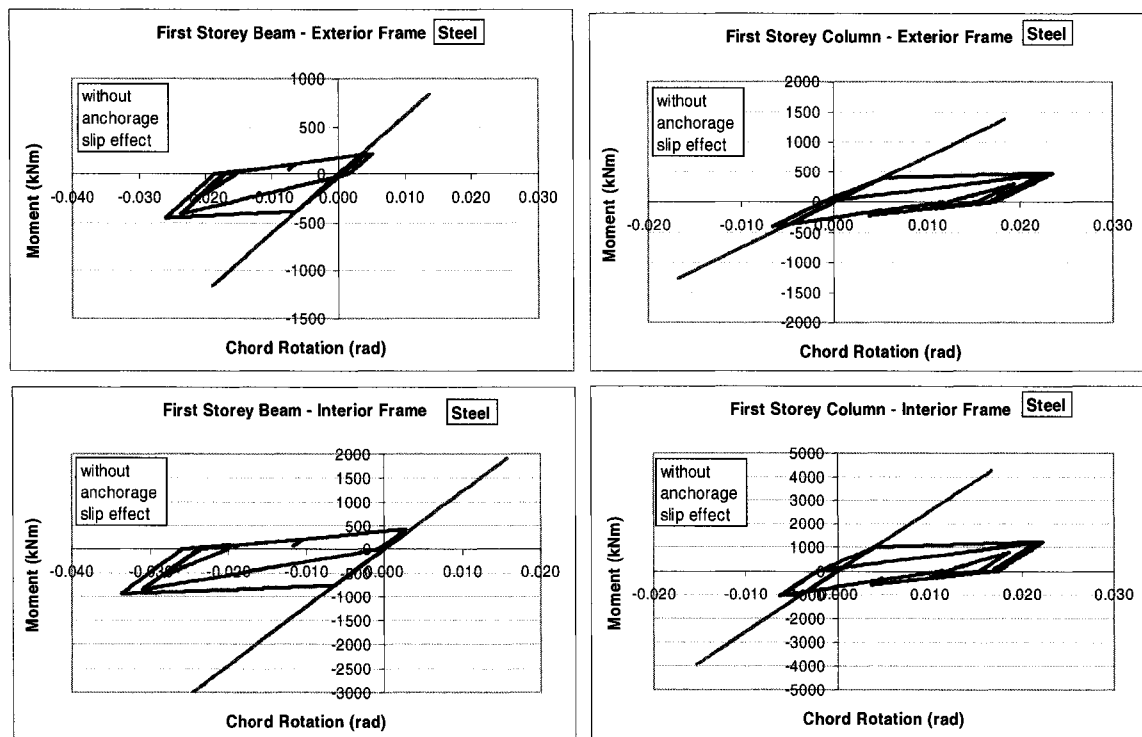


Figure 4.88 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver, without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Short Event 4-Vancouver' record

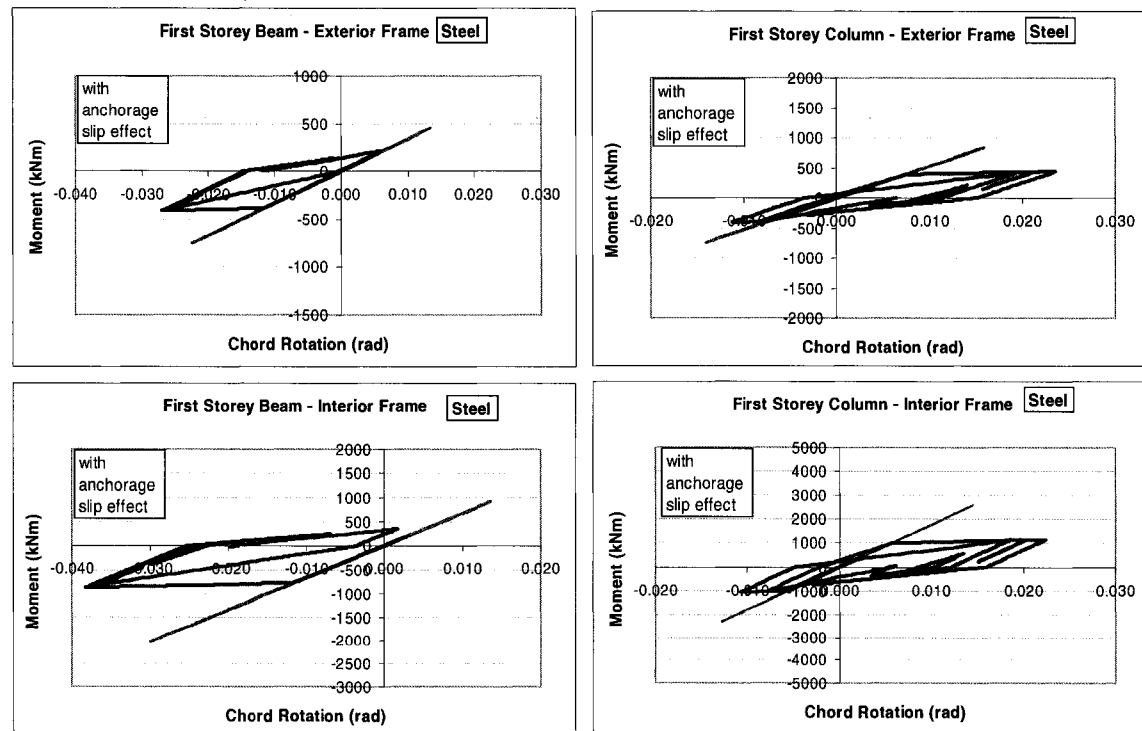


Figure 4.89 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver with anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Short Event 4-Vancouver' record

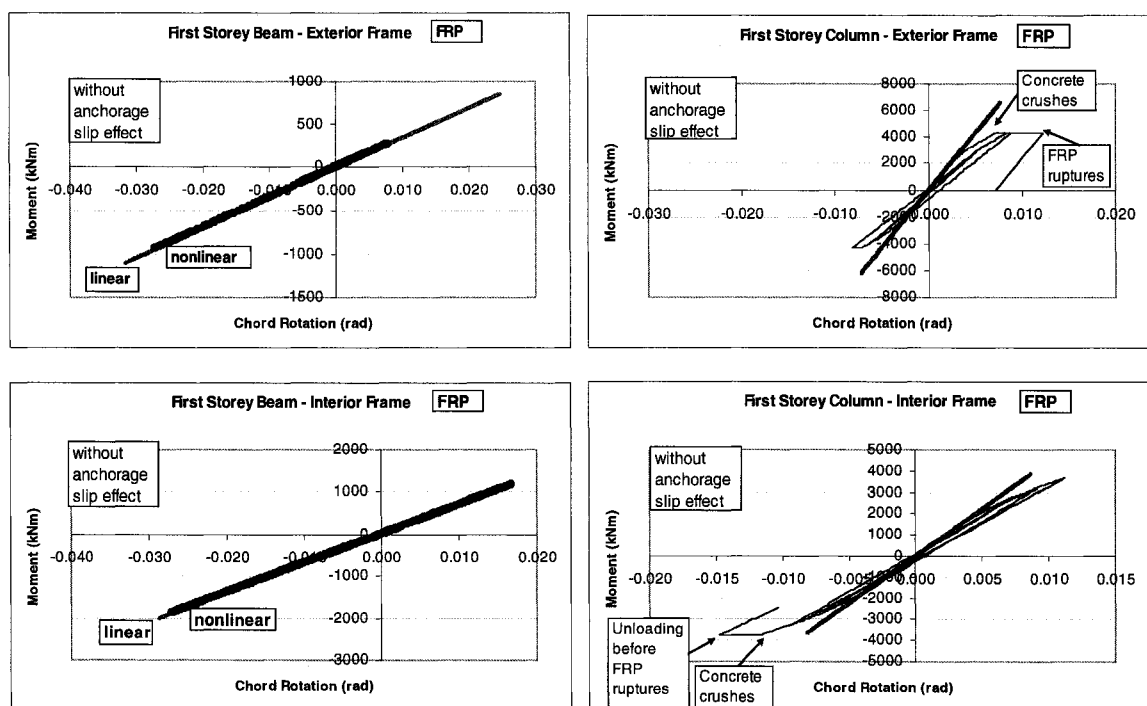


Figure 4.90 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Short Event 4-Vancouver' record

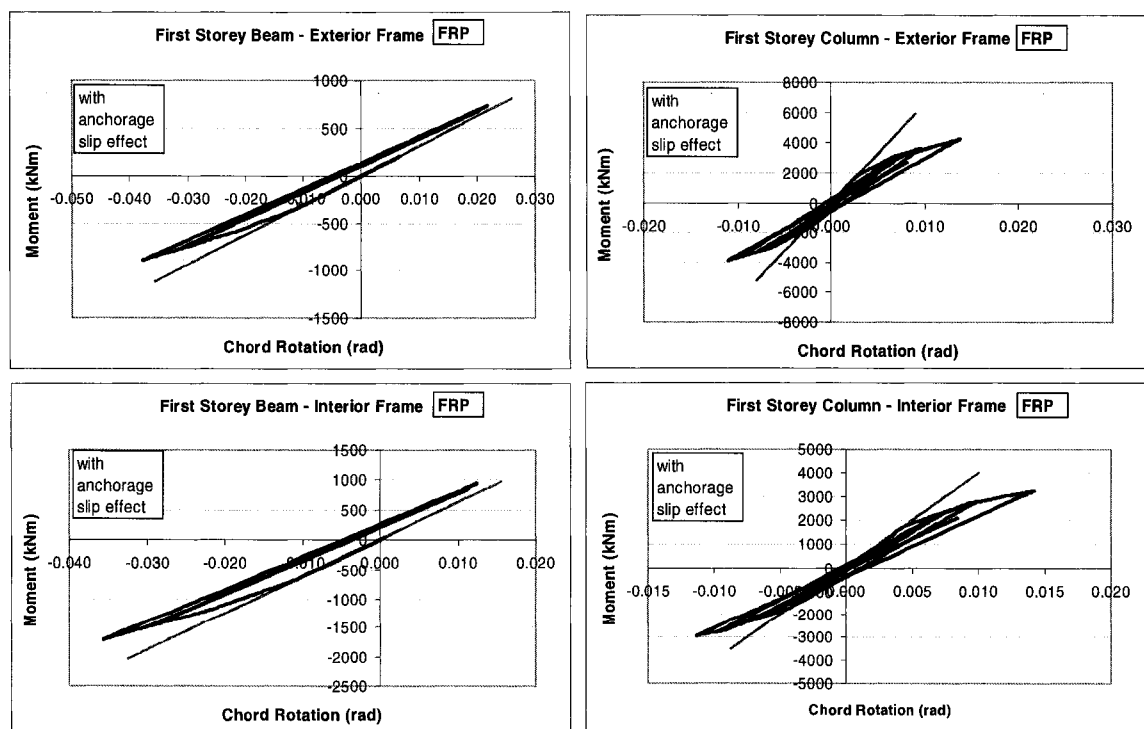


Figure 4.91 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver with anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Short Event 4-Vancouver' record

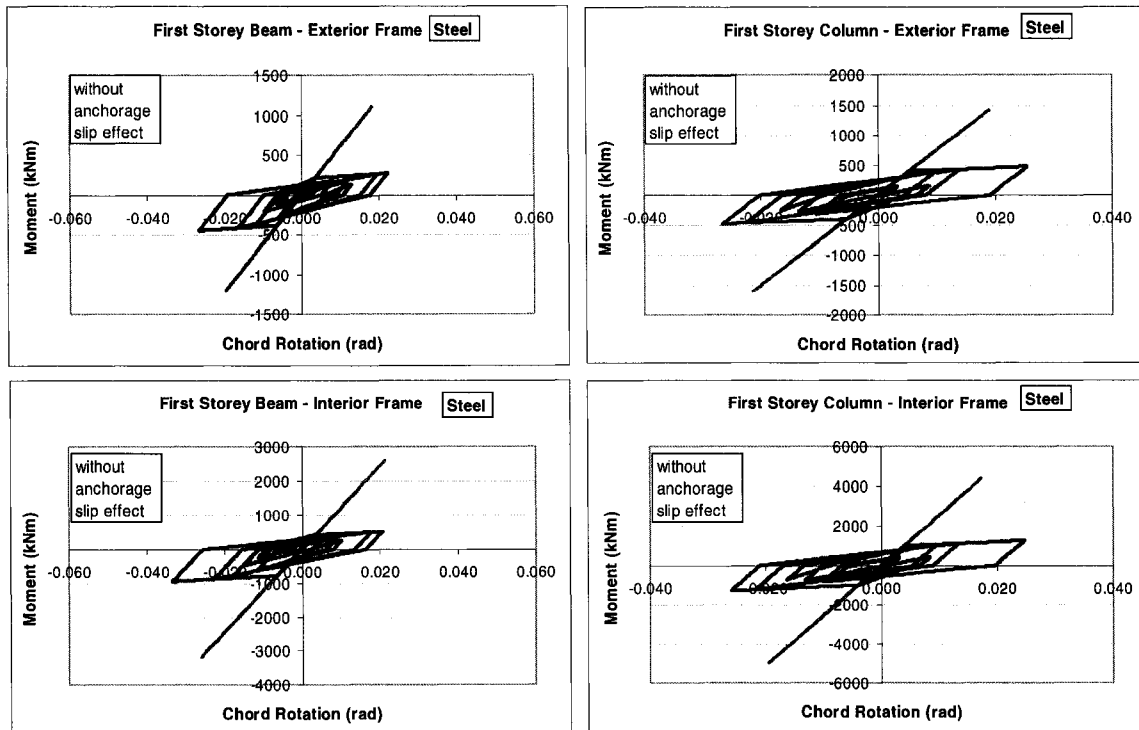


Figure 4.92 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Long Event 1-Vancouver' record

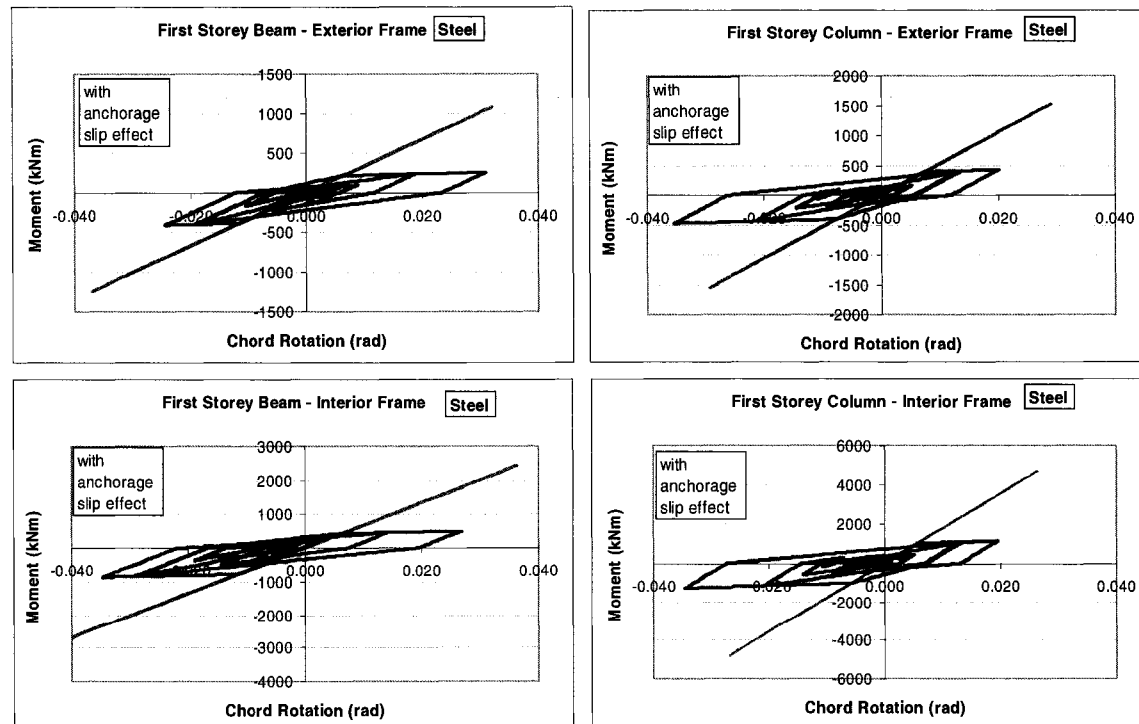


Figure 4.93 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver with anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Long Event 1-Vancouver' record

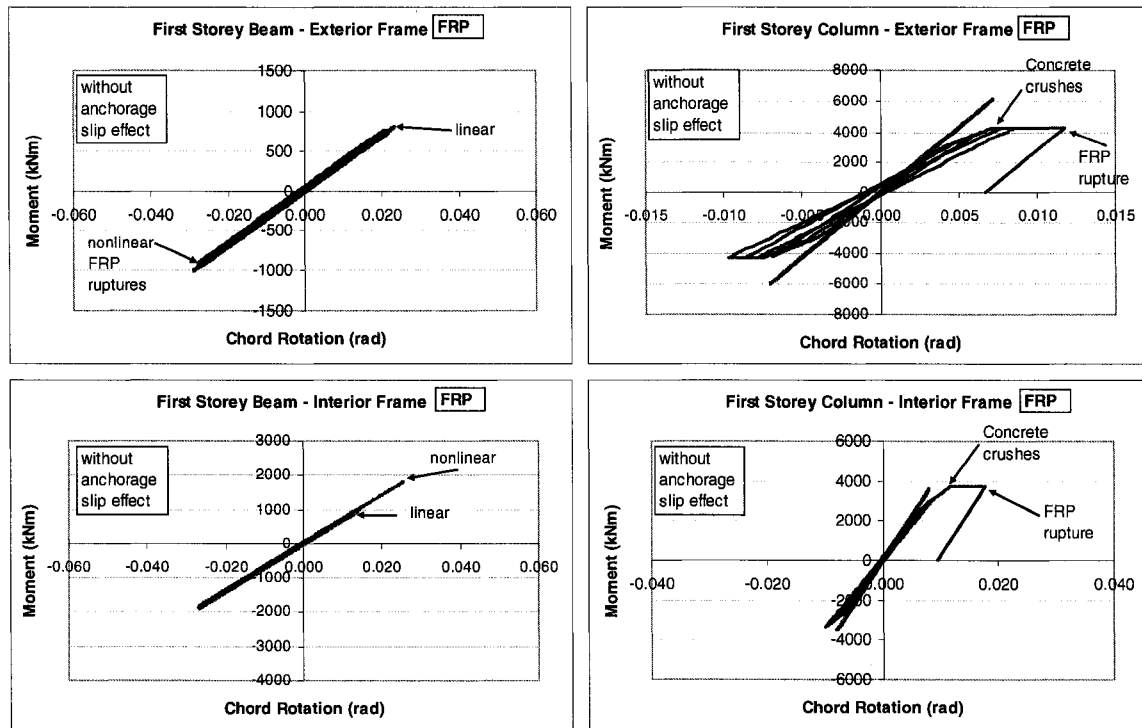


Figure 4.94 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Long Event 1-Vancouver' record

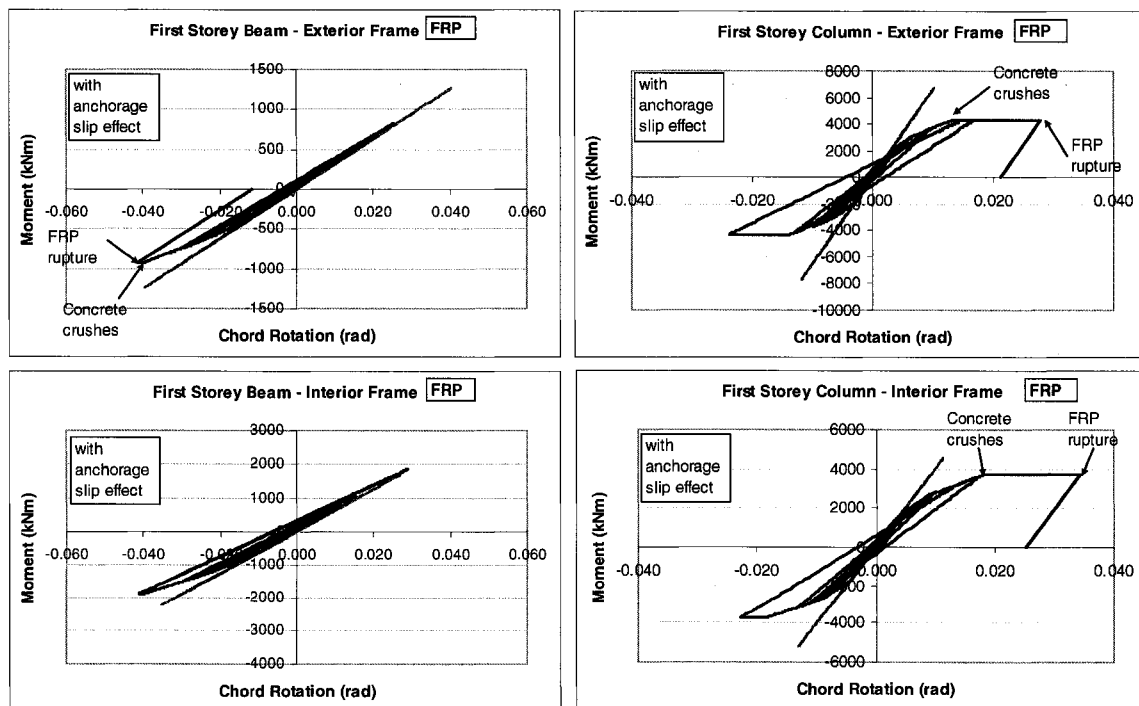


Figure 4.95 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver with anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Long Event 1-Vancouver' record

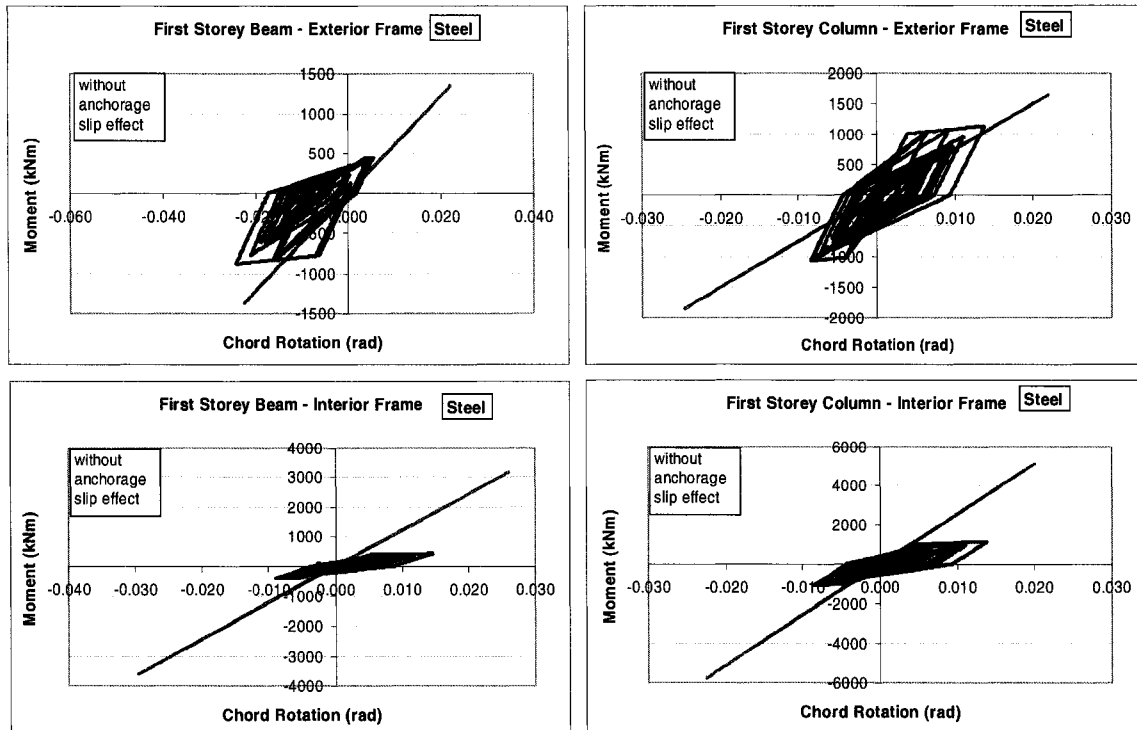


Figure 4.96 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Nisqually (2001)' record

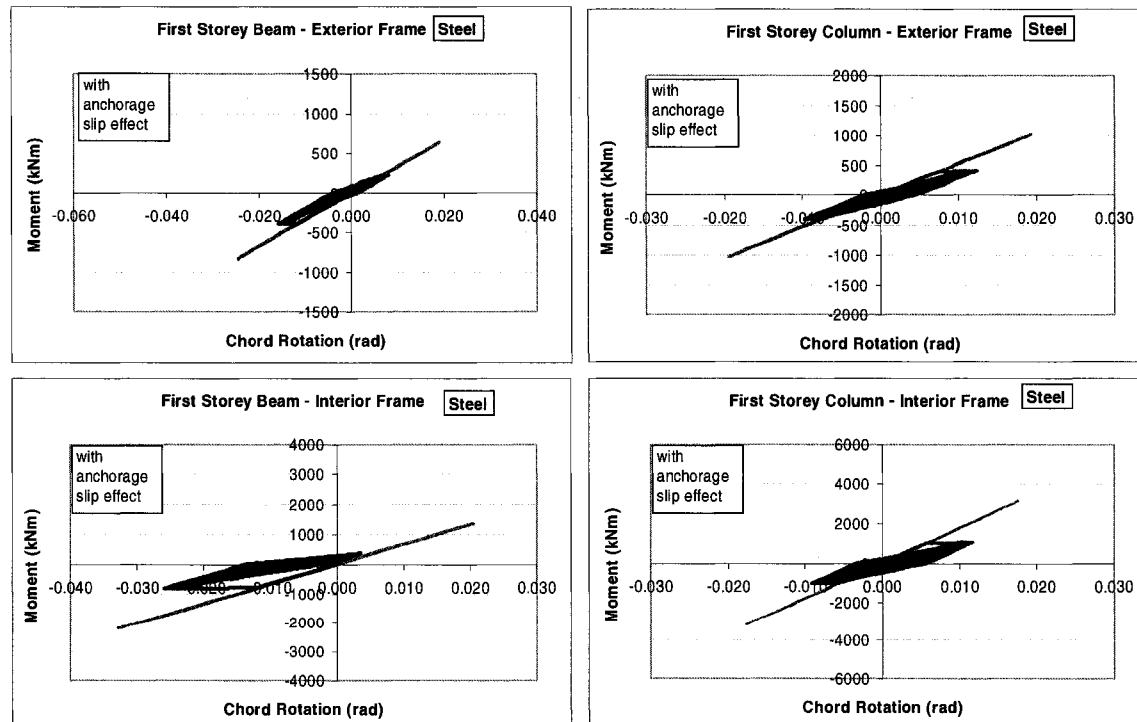


Figure 4.97 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey steel reinforced building in Vancouver with anchorage slip effect – linear and nonlinear analyses under the 100 % intensified 'Nisqually (2001)' record

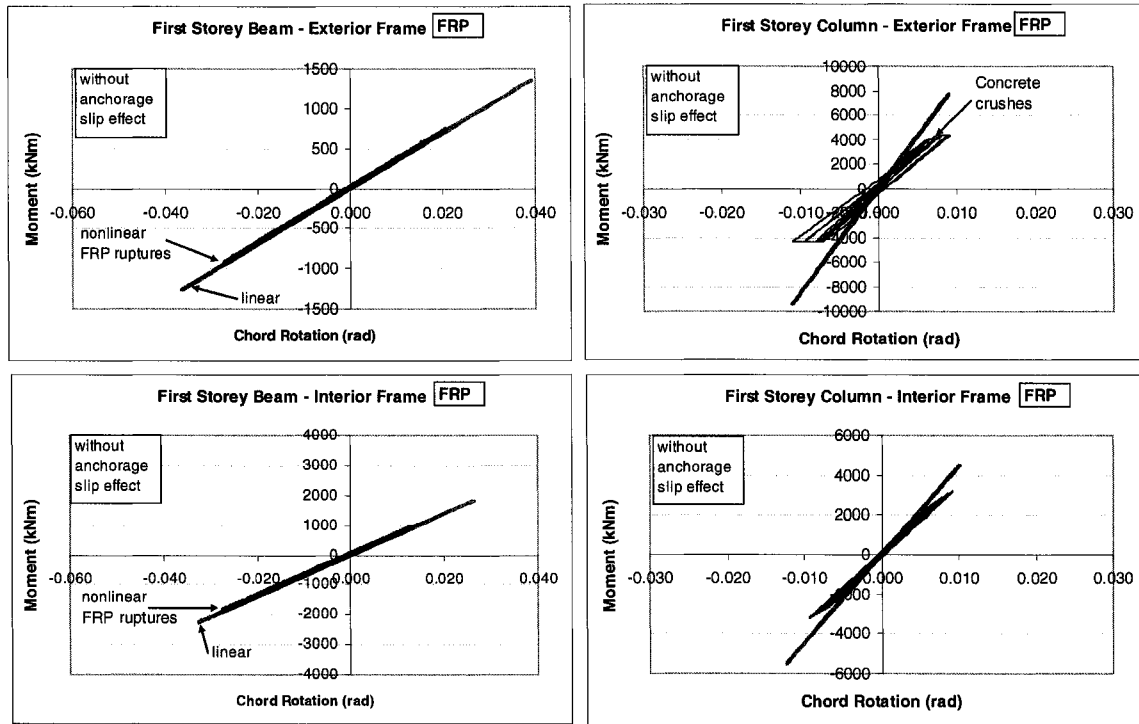


Figure 4.98 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver without anchorage slip effect - linear and nonlinear analyses under the 100 % intensified 'Nisqually (2001)' record

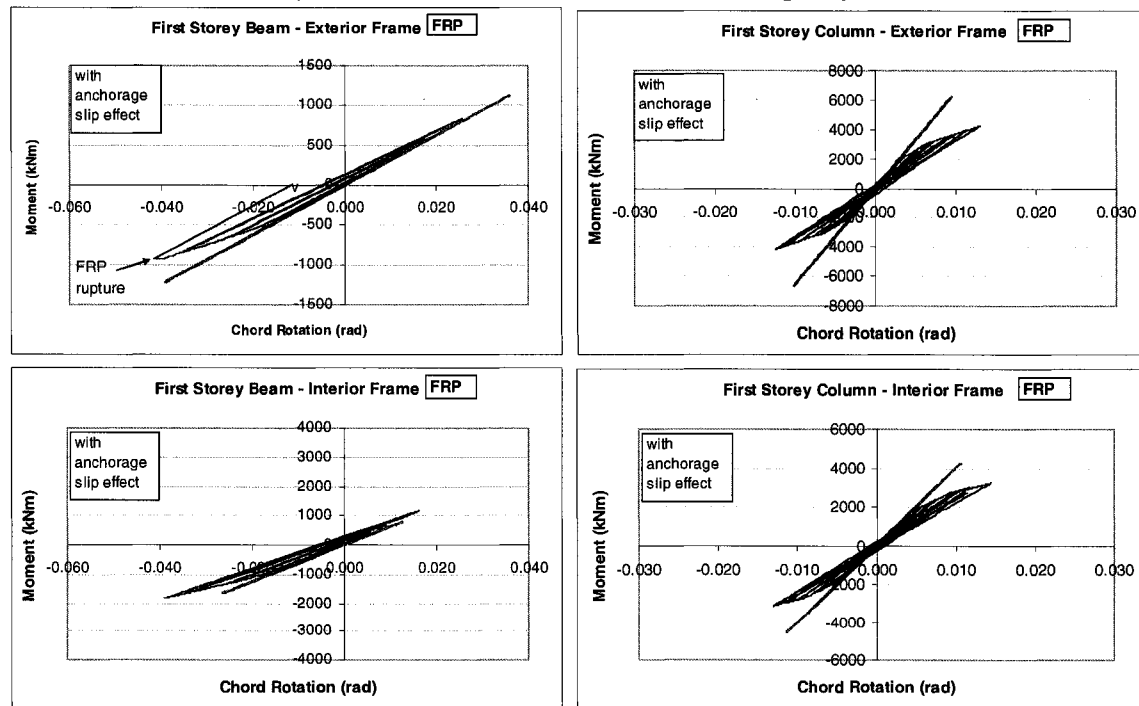


Figure 4.99 Moment-chord rotation hysteretic relationship for the beams and columns of 5 storey FRP reinforced building in Vancouver with anchorage slip effect, - linear and nonlinear analyses under the 100 % intensified 'Nisqually (2001)' record

Chapter 5

Design of FRP Reinforced Concrete Buildings for Seismic Effects

5.1 Introduction

An important outcome of the current research program is the generation of design information for FRP reinforced concrete buildings in seismically active regions. The current knowledge on the design and performance of FRP reinforced concrete buildings under ordinary gravity loads is limited. Significant research effort has been undertaken in the area during the last two decades, but much more is needed to increase the confidence of engineering profession for widespread use of FRP reinforcement in concrete structural elements. This becomes more critical for buildings in seismically active regions. The current seismic design practice, worldwide, is based on the concept of dissipating seismic induced energy, where possible, through significant inelasticity while maintaining strength, stiffness and stability of structures. The extension of this practice to FRP reinforced concrete structures creates many challenges. The use of two brittle materials, i.e., concrete and FRP, in producing ductile structural elements to dissipate earthquake energy requires extensive research, innovation and a shift in conventional thinking. Furthermore, the delicate balance between strength and deformation requirements, and the interaction between the dynamic properties of structures and seismic excitations need to be established. This requires the generation of extensive data through combined experimental and analytical research. Research at the University of Ottawa is geared towards this goal. While tests of full-size and near full-size FRP reinforced concrete elements are conducted under simulated seismic loading, dynamic inelastic response time history analyses of buildings are conducted to assess seismic demands. The current research has generated analytical design data for buildings in Canada, using the most recent Canadian seismic hazard information. This design data is used, in combination

with experimental observations obtained by others in different phases of the same overall research project, and a design approach is developed. The details of the approach are presented in this Chapter, following a brief summary of earthquake resistant design philosophy in Section 5.2 and a summary of results on experimental and analytical research providing information on seismic capacities and demands in Sections 5.3 and 5.4, respectively. Design aids are presented, when appropriate.

5.2 Performance Levels and Design Philosophy

It is generally accepted by the earthquake engineering profession that elastic seismic design forces, established with due considerations given to seismic hazards, local soil conditions and dynamic properties of structures, are reduced to account for available over-strength and ductility, with reasonable assurances to attain target performance levels. This implies that seismic design forces are established for certain performance levels, with due considerations given to deformation demands and capacities, and the associated damage levels.

The NBCC has maintained, over the years, “life safety” as the acceptable performance level for building design, with “operational (or immediate occupancy)” performance level assigned to post-disaster buildings. The qualitative descriptions used to express performance levels are quantified by researchers in a number of different ways. It has become a common practice in recent years to express descriptive performance levels and associated damage in terms of lateral drift ratios. The use of drift ratio captures not only potential damage associated with structural deficiencies but also incorporates damage resulting from poor performance of often brittle non-structural building elements and operational and functional components (OFCs). The FEMA-356 (2000) document provides a clear description of the relationship between drift levels and corresponding structural damage while emphasizing the approximate nature of this relationship in view of the inherent uncertainties associated with seismic risk. Accordingly, 2% drift limit is found to be appropriate to describe life safety performance level for reinforced concrete frame buildings. This level of drift is expected to cause extensive damage to beams while causing the spalling of cover concrete in ductile columns. Shear cracks are expected to be

visible, though limited to 3 mm in width. Similar diagonal cracks are expected in beam-column joints. Damage to non-structural elements is limited as they remain intact to ensure life safety.

Collapse prevention is attained at 4% lateral drift ratio. At this deformation level significant hinging is expected in ductile frame elements. Short columns suffer severe damage with occasional splice failure of non-ductile columns and buckling of compression bars. Very little reserve strength and stiffness remain in the structure, though the primary framing system continues to carry gravity loads with significant permanent deformations.

FEMA-356 (2000) document associates “immediate occupancy” performance level with a drift limit of 1%. At this deformation level only minor hairline cracks are expected with limited yielding of members taking place at few locations. No concrete crushing is anticipated. Some joint shear cracking is expected with crack widths remaining within 1.5 mm.

Although lateral drift in most building codes, including the NBCC-2005 is limited to 1.0% for post-disaster buildings, 2.0% for schools and 2.5% for all other buildings, the individual elements of seismic force resisting systems are designed and detailed for significantly higher drift levels. For example, reinforced concrete frames with full ductility are designed for a ductility related force reduction factor, $R_d = 4.0$. Because most flexure-dominant columns develop full yield displacement (beyond the initial sectional yielding of extreme layer of reinforcement) at approximately 1% lateral drift ratio, $R_d = 4.0$ translates into a drift level of approximately 4%, which is often associated with collapse prevention. For moderately ductile buildings the R_d factor specified in the NBCC-2005 is 2.5, corresponding to approximately 2.5% drift ratio. The R_d factor reduces to 1.5 for ordinary reinforced concrete buildings that do not have special seismic design and detailing. This is explained by the presence of some ductility in conventional reinforced concrete construction even without the special seismic detailing. The NBCC-

2005 recommends $R_d = 1.0$ for other concrete buildings which may not have any dependable ductility.

If the current design practice for steel reinforced concrete frames is to be extended to FRP reinforced concrete frame buildings, the above mentioned performance levels and associated drift limits should be considered. This is why the results of the current investigation play a crucial role in understanding drift demands as they relate to expected performance levels under different seismic hazard conditions.

5.3 Ductility Capacities in FRP Reinforced Concrete Elements

Ductility capacities of FRP reinforced concrete columns and beams were established by reviewing the results of large-scale tests conducted under simulated seismic loading by Sharbatdar and Saatcioglu (2004a, 2004b, 2005, 2007a, 2007b, 2007c) at the University of Ottawa, as part of the same overall project. Table 5.1 provides a summary of selected column tests conducted in the experimental program. The test columns consisted of specimens with different CFRP transverse reinforcement and correspondingly different concrete confinement, as well as different shear spans creating flexure-dominant and shear dominant responses. Table 5.1 also provides maximum drift ratios observed at 20% strength decay beyond the maximum force resistance. This level of strength decay is usually taken as an acceptable level of deterioration in multi-bay, multi-storey buildings where considerable redistribution of stresses occurs before a column is considered to have failed.

Figure 5.1 illustrates typical hysteretic relationships for columns with poorly and well confined concrete cores. Column CFCL1 with virtually no confinement provided by 4-cell CFRP grids of 0.27% area in each transverse direction (computed as $A_{FRP}/b_c s$ where A_{FRP} is the total FRP area in one cross sectional direction, b_c is the column core dimension measured centre-to-centre of perimeter hoops, and s is the grid spacing) and $h/2$ grid spacing showed 1% drift capacity. When the FRP reinforcement ratio was increased to 0.37% with an improved grid arrangement of 9-cells, Column CFCL3 was able to develop approximately 2% drift ratio. However, when the grid spacing was

reduced to $h/4$, Column CFCL2 with 0.55% 4-cell FRP grids and Column CFCL4 with 0.73% 9-cell FRP grids developed approximately 3% to 4% drift capacities. In addition to these flexure-dominant columns with long shear-spans a number of columns with short shear-spans were tested under high shear force reversals. These columns, when designed for appropriate amounts of FRP shear reinforcement also showed inelastic deformability. An example of their hysteretic behaviour is illustrated in Fig. 5.1(d), which had 0.73% 9-cell grids spaced at $h/4$ and developed 2% to 3% lateral drift prior to significant strength decay. This indicates that some ductility can be expected from FRP reinforced concrete elements, even if the constituent materials, i.e., concrete and FRP, both show brittle characteristics.

Although the column deformation capacity may be viewed as the governing factor for inter-storey drift capacity of a frame building with rigid floors one needs to assess the deformability of attached beams. Sharbatdar and Saatcioglu (2005a) also tested 6 large-scale beams under either monotonically increasing or reversed cyclic loading. The beams had short or long shear spans with shear or flexure dominant responses. The results indicated that the behaviour of FRP reinforced concrete beams is different than that of columns. The column behaviour under axial compression is affected significantly by the characteristics of concrete. Hence, concrete confinement results in improved inelastic deformability (ductility) of columns. This is not true for reinforced concrete beams. Beams develop very small compression zones near ultimate load. Therefore, their behaviour is predominantly governed by reinforcement. Figure 5.2 shows sample force-deformation relationships recorded experimentally. It indicates that FRP reinforced concrete beams perform essentially elastically, with little or no inelastic deformability due to concrete confinement. This was observed to be true for beams designed to be over-reinforced (ultimate concrete strain attained prior to the FRP failure) as well. The beams, however, develop appreciable drift ratios (chord angles), because of reduced elastic rigidities associated with the use of low modulus CFRP bars and lack of axial compression and associated stiffening effects. The results show approximately 2% to 4% drift capacity within the elastic range of deformations, indicating sufficient deformability to accommodate potential rotational demands in flexible floors.

5.4 Force and Deformation Demands for FRP Reinforced Concrete Elements

Seismic demands on FRP reinforced concrete buildings were established through dynamic inelastic analysis of selected structures under UHS compatible earthquake records. Three different heights of frame buildings (5, 10 and 15-storey) were considered in the current investigation to investigate seismic demands in two Canadian cities (Ottawa and Vancouver) which have two different levels of seismicity (medium and high) representing eastern and western seismicity. Each building was designed twice, first using steel reinforcement and secondly using CFRP reinforcement. Because of drift limits imposed by NBCC-2005 and the variations in designs using the two types of reinforcement, some variations were introduced to member sizes and span lengths.

The FRP reinforced concrete frame buildings considered in this investigation were all designed for elastic response ($R_d R_o = 1.0$). This was done because of; i) lack of seismic design procedures promoting inelasticity in FRP reinforced structural components and limited experimental data indicating the possibility of a ductility based reduction in design base shears, ii) brittle characteristics of FRP and sudden rupturing of FRP reinforcement resulting in immediate loss of strength, and iii) inherent uncertainties associated with earthquake engineering. Therefore, the design base shears for FRP reinforced buildings, summarized in Table 4.1 of Chapter 4, are significantly higher than those used for designing steel reinforced concrete frames. Non-linear response was investigated under increased seismic hazards. Softening effects of lower modulus FRP reinforcement and anchorage slip within the adjoining members were investigated to assess seismic force and deformation demands. The following sub-sections provide a discussion of base shear and deformation demands for FRP reinforced concrete frame buildings.

5.4.1 Seismic Base Shear Demands

Seismic base shear demands were computed through dynamic response history analyses under NBCC-2005 compatible earthquake records. The computed shear demands were

significantly lower than the design base shear forces established through equivalent static load analysis. In Ottawa (representing medium seismic hazard in eastern Canada), the buildings remained elastic with maximum base shear values of 21% to 28% of elastic design forces, indicating that the buildings were over-designed using the elastic force levels stipulated in NBCC-2005. A similar trend was observed for steel reinforced concrete buildings designed for Ottawa. In this case, although the buildings were designed for lower inelastic design forces ($R_d = 2.5$ and $R_o = 1.4$), they remained elastic during dynamic response and developed shear force demands of 70% to 95% of their inelastic design force levels. The difference in shear demands established by dynamic analyses and equivalent static code values can be attributed to a large extent to differences in building periods established through empirical code expression and dynamic analysis, with the latter being significantly longer, reducing seismic shear demands. Also, although NBCC-2005 compatible earthquake records were employed in the analyses, the match between response spectra for these records and the corresponding UHS was not perfect, resulting in variations in base shear values.

The analyses of FRP reinforced concrete buildings in Vancouver showed a similar shear demand trends as those analyzed for Ottawa. Shear force demands established through dynamic analyses were 53% to 71% of the elastic design forces computed based on NBCC-2005.

The analyses conducted to establish seismic force demands were extended to incorporate the effects of anchorage slip of longitudinal reinforcement within adjoining members. This was of interest because of the difference in material stress-strain characteristics between FRP and conventional steel reinforcement. The effect of anchorage slip is to introduce softening in structural stiffness and elongation of vibration periods. The comparative results tabulated in Table 4.3 indicate that the consideration of anchorage slip can result in appreciable differences in seismic force demands (about 30% reduction was observed in the 5-storey FRP reinforced concrete building designed for Vancouver). Anchorage slip elongates fundamental period and may reduce seismic force demands, though, this depends on the spectral shape for a given seismic record and may

occasionally result in increases in response due to the interaction of pique points of the record. The analyses results indicated that anchorage slip should be considered in establishing effective elastic stiffnesses of structural members and the fundamental period. It should also be considered when modeling members for dynamic analysis.

The FRP reinforced buildings were designed to perform elastically and they behaved elastically in dynamic analysis. The effect of inelasticity was investigated under intensified earthquake records. The results show that it is possible to reduce seismic force demands through inelasticity while keeping inelastic deformations within experimentally observed limits, as discussed in Chapter 4. This observation is significant in terms of adopting R_d and R_o factors that are higher than 1.0.

5.4.2 Drift Demands

Drift demands for FRP reinforced concrete buildings were established both through static analysis under elastic seismic design forces and dynamic time history analyses under code compatible earthquake records. The results, summarized in Table 4.2, indicate that maximum drift ratios obtained by equivalent static load analyses range between 1.3% and 1.5% for buildings in Ottawa; and 1.9 and 2.1 for buildings in Vancouver. These maximum drift demands indicate that the buildings meet the life-safety performance criterion stipulated in the NBCC-2005. The drift demands established through dynamic time history analyses are lower than those computed statically, showing a range that varies between 0.2% to 0.4% for buildings in Ottawa; and 1% and 1.4% for buildings in Vancouver.

The drift demands for FRP reinforced concrete buildings are comparable to those obtained for steel-reinforced concrete buildings, implying that similar performance levels can be attained during moderate to strong earthquakes. The steel reinforced concrete buildings in Ottawa showed inelastic drift demands of 1.1% to 1.4% under code specified static design forces and 0.2% to 0.4% when subjected to code compatible earthquake excitations, as indicated in Table 4.2. The drift demands for steel reinforced buildings in Vancouver showed 2.0% to 2.4% drift under code specified static design forces and 0.8%

to 1.5% when subjected to code compatible earthquake excitations, as depicted in Table 4.2.

5.5 Recommendations for Seismic Design of FRP Reinforced Concrete Frame Buildings

5.5.1 Seismic Design Forces

Seismic design forces specified in NBCC-2005 for steel reinforced concrete frame buildings are also applicable to FRP reinforced concrete buildings, with appropriate modifications. The factors that contribute to seismic design forces that are independent of structure type and materials, including the Canadian seismicity (hazard), foundation conditions, higher mode effects, building importance and torsional considerations remain the same as outlined in the code. However, additional provisions are needed for FRP reinforced concrete buildings, as described in the following subsections.

5.5.1.1 Equivalent Static Force Approach

The design base shear force for FRP reinforced concrete frame buildings can be established empirically using the following NBCC-2005 expression for equivalent static base shear:

$$V = \frac{S(T_a) M_v I_E W}{R_o R_d} \quad (5.1)$$

Provided that buildings are;

- Located in low-seismic zones ($I_E F_a S_a(0.2) < 0.35$)
- Regular as defined in NBCC-2005 for which building height $h < 60$ m and $T_a < 2$ s
- Irregular as defined in NBCC-2005 for which building height $h < 20$ m and $T_a < 0.5$ s and are not torsionally sensitive.

In Eq. 5.1; $S(T_a)$ is the design spectral acceleration for fundamental period T_a , M_v is a factor that reflects higher mode effects, I_E is building importance factor, and W is the total

weight of structure, consisting of dead loads and a portion of live loads, as per NBCC-2005. R_d and R_o are ductility and over-strength related force modification factors, discussed in Section 5.5.1.2.

The equivalent static base shear for buildings with fundamental periods longer than 2.0sec is specified as a constant value fixed at $T_a = 2.0$ s. The upper limit for base shear is limited for ductile structures with $R_d \geq 1.5$ to:

$$V = \frac{2}{3} \frac{S(0.2) I_E W}{R_o R_d} \quad (5.2)$$

The individual terms used in Eqs. 5.1 and 5.2 are described in NBCC-2005 and their values are also specified in the same code. Therefore, they are not repeated here unless modified for FRP reinforced concrete buildings, in which case the revised values are presented.

The base shear computed by Eq. 5.1 is distributed along the height of the structure as prescribed in NBCC-2005. Elastic analysis results under these equivalent static loads provide design forces and deflections. The elastic deflections obtained (under reduced inelastic design force levels) need to be multiplied by force reduction factors, $R_d R_o$ to obtain inelastic deflection demands.

The fundamental period T for steel reinforced concrete frame buildings can be computed based on the empirical expression specified in NBCC-2005, shown below.

$$T = 0.075 (h_n)^{3/4} \quad (5.3)$$

The above expression was established based on field measurements of steel reinforced concrete frame structures. It incorporates the stiffening effect of non-structural elements that may be present in the structure. Unfortunately, field data is not available for such

buildings. Therefore, until further research findings become available it is recommended to use Eq. 5.3 for FRP reinforced concrete frame buildings as well.

More accurate estimation of fundamental periods of structures can be obtained by using the accepted methods of mechanics (ex: Eigen value analysis using the computer software SEQUAKE). The effective elastic stiffness for such analysis can be taken as $0.2EI_g$ and $0.5EI_g$ for FRP reinforced concrete beams and columns, respectively (as opposed to $0.35EI_g$ and $0.7EI_g$ specified by CSA A23.3-04 for steel reinforced beams and columns, respectively). The rationale for this recommendation is discussed in Section 3.2.1. When the fundamental period for design is computed by using the accepted methods of mechanics, the value used for design should not exceed 1.5 times the value computed by the empirical expression given in Eq. 5.3. This limit is also specified in NBCC-2005 for steel reinforced concrete frame buildings.

5.5.1.2 Ductility and Over-strength Related Force Modification Factors

Structures with dependable ductility and over-strength are designed for reduced seismic forces relative to their elastic seismic demands. This is done to take advantage of available ductility and over-strength while dissipating seismic induced energy through inelasticity. The force reduction factors are incorporated in Eq. 5.1. This equation defines inelastic design base shear requirement for earthquake resistant buildings. Ductility of FRP reinforced concrete elements is a controversial concept since the constituent materials, both concrete and FRP are brittle materials. However, recent experimental research conducted by Sharbatdar and Saatcioglu (2007a, 2007b, 2007c) and summarized in Section 5.3 indicates that well confined FRP reinforced concrete columns can develop ductility while the beams remain essentially elastic. Based on the examination of experimental results, the following R_d factors are recommended for seismic design of FRP reinforced concrete frame buildings:

$R_d = 1.0$ for all FRP reinforced concrete buildings, except

$R_d = 2$ for frame buildings with columns designed to confirm the confinement requirements of Section 5.3.2.

Over-strength related force modification factor for steel reinforced concrete buildings can be computed by considering a number of parameters as indicated below:

$$R_o = R_{size} * R_{\phi} * R_{yield} * R_{sh} * R_{mech} \quad (5.4)$$

R_{size} relates to member sizes. Practical design considerations often lead to conservative rounding of element sizes because of construction and functional requirements. Furthermore, the use of standard size reinforcing bars results in some over-strength. Size related over-strength has been found to result in approximately 5% increase in member strength (Mitchell et al. 2003). Therefore, $R_{size} = 1.05$ may be appropriate for FRP reinforced concrete elements as well.

R_{ϕ} relates to material resistance factors (ϕ) that are employed in structural design. The ϕ factor provides a safety margin in design. This additional safety margin may be removed because of the extremely rare nature of seismic hazard (with a return period of 2500 years). The ϕ factor to be removed should be based on the predominant material that governs the particular mode of behaviour that is of concern. For steel reinforced concrete elements the member design is based on under-reinforced element performance where steel yielding is promoted prior to the crushing of concrete. Hence, $R_{\phi} = 1/\phi = 1/0.85 = 1.18$. However, FRP reinforced concrete elements are designed to be over-reinforced because of the abrupt nature of FRP rupture. This implies that the dominant material that affects element strength is concrete. Therefore, for FRP reinforced concrete elements, $R_{\phi} = 1/\phi = 1/0.65 = 1.5$.

R_{yield} and R_{sh} refer to over-strength associated with the use of higher than specified yield strength and the presence of strain hardening in steel. Both of these conditions do not apply to FRP reinforced concrete. Therefore, they should be taken as 1.0. R_{mech} accounts for additional strength gain that may result from redundancies and redistributions of stresses prior to the formation of collapse mechanism. This redistribution and associated over-strength may not be dependable in FRP reinforced concrete buildings because of the

limited inelastic deformability of FRP reinforced concrete elements. Therefore, it should conservatively be taken as 1.0. Hence the following defines the over-strength related force modification factor R_o for FRP reinforced concrete buildings:

$R_o = 1.0$ for all FRP reinforced concrete buildings, except

$R_o = R_{size} * R_{\phi} = 1.05 * 1.50 = 1.60$ for frame buildings with columns designed to confirm the confinement requirements of Section 5.3.2.

5.5.1.3 Dynamic Analysis Approach

NBCC-2005 specifies dynamic analysis as the preferred approach. Dynamic analysis approach has become more convenient for use in design, with the availability of uniform hazard spectra (UHS) specified for all cities in Canada. Dynamic analysis for FRP reinforced concrete buildings can be conducted either in the form of elastic or inelastic analysis. The elastic dynamic analysis can be conducted either as modal response spectrum analysis or time history analysis. The procedure followed for any of these analysis techniques is the same for FRP reinforced and steel reinforced concrete buildings.

The principles of dynamic analysis are well established. However, the use of dynamic analysis as a design tool may be prone to modeling and interpretation inaccuracies. Therefore, NBCC-2005 places a limit on its use in establishing seismic force demands. Accordingly, for regular buildings, the base shear computed through dynamic analysis can not be more than 20% lower than that computed through the use of equivalent static load analysis. For irregular buildings, design base shear computed through dynamic analysis should not be taken lower than that established by equivalent static analysis. Once the analysis results are obtained, the elastic based shear need to be reduced by $R_d R_o$ factors. Furthermore, the building importance factor " I_E " needs to be applied, as shown in the following expression.

$$V_d = \frac{V_e}{R_o R_d} I_E \quad (5.5)$$

Where, V_d and V_e are inelastic design and elastic base shear values, respectively. Although dynamic base shear values are tied to the static base shears obtained by equivalent static loads, it is permissible to use dynamic analysis results to obtain elastic deflections. These elastic deflection values are then increased by $R_d R_o$ to attain inelastic deflections.

Inelastic time history analysis, though not very common in a design office environment, can also be employed to establish seismic demands. In this case the structural modeling gains another level of complexity and importance. Therefore, a “special study” is required by NBCC-2005 for buildings designed using dynamic inelastic analysis. This is especially true for FRP reinforced concrete buildings for which there is limited information available. Currently the hysteretic model developed by Sharbatdar and Saatcioglu (2007d) is the only model that is available to describe loading, unloading and reloading stiffnesses of FRP reinforced concrete elements during dynamic inelastic response. This hysteretic model has been incorporated into the computer program SEQUAKE developed as part of the current research program. Hence, SEQUAKE can be employed as an analysis tool for design. Before dynamic inelastic analysis is employed for design, however, it is absolutely necessary to find computer software that is equipped with relevant hysteretic models. Unlike elastic dynamic analysis, inelastic analysis does provide inelastic design force levels directly. Hence, the results should not be modified further by $R_d R_o$ factors. Furthermore, structural importance factor “ I_E ” can be implemented indirectly either through adopting different levels of stringency for drift limits and/or through the use of different probability levels in selecting design ground motions for different levels of performance.

5.5.2 Design of FRP Reinforced Concrete Elements for Seismic Loads

Designing concrete structures with FRP reinforcement is a new concept for structural engineers. The basic principles are currently being developed and presented as guidelines and standards (CSA S806-02, 2002, ISIS Manual, 2001). These documents reflect the state-of-the-art in design while lacking some aspects of design, including the design of FRP reinforced concrete columns and design for seismic forces. This section presents

some of the missing design information developed as part of the current investigation. They are intended to complement existing design procedures.

5.5.2.1 Design of FRP Reinforced Concrete Columns Subjected to Combined Bending and Axial Force

Little information is available on the performance of FRP reinforced concrete elements under combined bending and axial load. Some experimental research was conducted on columns under monotonically increasing compression (Wu 1990, Kobayashi and Fujisaki 1995, Paramanantham 1993, Alsayed et. al 1999, Sonobe et al 1997, Sharbatdar and Saatcioglu 2007a). A few test programs have also been conducted under reversed cyclic loading (Fukuyama and Masuda 1995, Sharbatdar and Saatcioglu 2007b, 2007c). The results reveal that FRP reinforced concrete columns can be designed using standard plane section analysis. However, the analysis should reflect the stress-strain relationships of materials, especially of the FRP reinforcement in compression. The stress-strain characteristics of FRP reinforcement in tension can be determined by coupon tests and show linearly elastic behaviour. Unlike steel reinforcement, the stress-strain characteristics in compression show different behaviour. Both the elastic modulus and compressive strength decreases significantly and this should be incorporated in the analysis. According to Saatcioglu and Sharbatdar (2001) the strength of FRP reinforcement can be as low as approximately 20% of its tensile strength. Furthermore, the elastic modulus in compression can be decreased to approximately 20% of its modulus in tension. These characteristics of FRP longitudinal reinforcement should be incorporated in sectional analysis of columns.

It is convenient to design FRP reinforced concrete columns by using P-M Interaction Diagrams, as is the case for steel-reinforced concrete columns. However, these interaction diagrams are currently not available for use in design. Therefore, a set of interaction diagrams were developed for FRP reinforced concrete columns as part of the current research program. This was done by following the standard procedure while recognizing that the rupturing of FRP as a failure mode may not be acceptable and the columns may have to be over-reinforced within the entire range of axial compression.

Accordingly, the crushing of concrete in the extreme compression fibre was assumed to occur at 0.0035 strain (as per CSA A23.3-2004) and axial load capacity “P” and bending moment capacity “M” were computed for different values of neutral axis location “c.” This was done for square columns with equal reinforcement on all sides. The resulting diagrams are shown in Figs. 5.3. The diagrams were developed for concrete strengths of $f'_c = 30\text{MPa}$, 40MPa , 60MPa and 80MPa and FRP strength of $f_{FRP} = 1560\text{ MPa}$. Two different elastic moduli were used for FRP bars, since the fibre content and the type of fibre can have bearings on elastic modulus; these were $E_{FRP} = 100,000\text{MPa}$ and $200,000\text{MPa}$. A linear extrapolation can be made for in-between values. Two different concrete cover values were used, as incorporated through γ (the ratio of centre to centre distance between perimeter bars at two opposite faces to column dimension). The diagrams were developed for axial load and moment resistances (P_r and M_r), computed by applying the concrete material resistance $\phi_c = 0.65$ and the FRP material resistance factor $\phi_{FRP} = 0.75$.

5.5.2.2 Design of FRP Reinforced Concrete Columns for Confinement

The experimental evidence to date (Sharbatdar and Saatcioglu, 2004, 2005, 2007a, 2007b) albeit limited, indicate that concrete columns reinforced with CFRP bars as longitudinal reinforcement and CFRP grids as transverse ties can develop some inelasticity in flexure through the confinement of compression concrete. This is similar to steel reinforced concrete columns subjected axial compression above the balanced load when the compression concrete develops crushing strains prior to the yielding of steel reinforcement in tension.

A displacement-based design procedure was developed earlier by Saatcioglu and Razvi (2002) for steel reinforced concrete column confinement. The procedure was adopted by ACI ITG-4.3R-07 (2007) document, entitled “Report on Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications.” It was also adopted by CSA S806-02 (2003) “Design and Construction of Building Components with Fibre-Reinforced Polymers.” In the latter document, however, the application was limited to FRP transverse reinforcement confining concrete columns with steel longitudinal bars.

This limitation was specified because at the time of the development of the standard, there was not enough data available on the performance of concrete columns reinforced with FRP longitudinal reinforcement. The main difference between concrete columns reinforced with steel and FRP longitudinal bars, confined with FRP transverse reinforcement is the behaviour of longitudinal bars in compression within the concrete compression zone. It was observed experimentally that longitudinal FRP bars fail in compression through the buckling of individual fibres at approximately 0.39 % to 0.55% compressive strains (Sharbatdar and Saatcioglu 2007b). This limits the axial strain deformability and effectiveness of transverse FRP reinforcement. Figure 5.18 illustrates the failure of FRP and steel bars in column compression regions when columns are confined with FRP grids. The compression steel bars can sustain significantly higher strains before they buckle. FRP bars sustain limited compressive strains before they fail. The columns experience strength decay prior to the full utilization of transverse reinforcement capacity as confinement reinforcement. Hence, FRP transverse reinforcement is not fully effective in FRP reinforced concrete columns, developing only a fraction of its strength.

The applicability of the previously developed confinement design procedure (Saatcioglu and Razvi 2002) to FRP reinforced concrete columns was verified with due considerations given to experimentally measured transverse FRP strains. The procedure was developed based on the computation of column drift capacity analytically while accounting for concrete confinement (Saatcioglu and Razvi 2002, Yalcin and Saatcioglu 2000, Razvi and Saatcioglu 1999). Accordingly, the following expression provides a relationship between column confinement parameters, the level of axial compression and the lateral drift capacity.

$$\rho_c = 14 \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P}{P_o} \delta \quad (5.6)$$

Where;

$$k_c = 0.15 \sqrt{\frac{b_c}{s} \frac{b_c}{s_l}} \leq 1.0 \quad (5.7)$$

$k_c = 1.0$ for closely spaced circular hoops and spirals.

$$\frac{P}{P_o} \geq 0.2 \quad (5.8)$$

$$\frac{A_g}{A_c} - 1 \geq 0.3 \quad (5.9)$$

A_c : Area of core concrete enclosed by centre-to-centre distances of perimeter ties.

A_g : Gross area of column section.

b_c : Core dimension centre-to-centre of perimeter tie.

f'_c : Concrete cylinder strength.

f_{yh} : Maximum stress developed in transverse reinforcement prior to column failure.

This value is equal to the yield strength of transverse steel for normal grade steel reinforcement.

k_c : Confinement efficiency parameter defined in Eq. 5.7.

P : Maximum axial compressive force on column.

P_o : Column concentric capacity.

s : Spacing of transverse confinement reinforcement along the column length.

s_l : Centre-to-centre spacing of longitudinal reinforcement laterally supported by the corner or hook of a tie.

δ : Drift ratio. Lateral displacement divided by column height.

ρ_c : Are ratio of confinement reinforcement. Total area of transverse reinforcement in one cross-sectional direction divided by the perpendicular concrete core dimension and the spacing “s” of ties.

The verification of the procedure was done by computing inelastic drift capacities of columns whose hysteretic behaviours are shown in Fig. 5.1. The lateral drift ratio “ δ ” was solved from Eq. 5.6 for each column tested. The maximum stress in transverse FRP reinforcement was taken as $0.006E_{FRP}$ ($0.006 \times 76,335 \text{ MPa} = 458 \text{ MPa}$) because during testing the strain gauge readings indicated up to 0.65% strain with a wide range of recorded strain values between 0.30% and 0.65%. Unfortunately, strain gauges often

provide localized strains in reinforcement and can not be taken as accurate representation of maximum strains developed in transverse reinforcement, especially in square grids because they do not develop uniform hoop tension as in the case of circular hoops or spirals. Earlier research on lightly loaded bridge columns conducted by Elnabehy and Saatcioglu (2004) indicated no more than 0.4% maximum strain in FRP transverse reinforcement when the level of axial compression was limited to 10% P_o . At the other extreme, Ozbakkaloglu and Saatcioglu (2006) tested columns encased in FRP tubes (well confined) under heavy axial compression and recorded tremendous increases in FRP strains up to the material rupturing strain. Saatcioglu et al (2008) recently recommended the use of a range of strain values for FRP depending on the level of axial compression and the effectiveness of confinement reinforcement as governed by the confinement efficiency parameter specified in Eq. 5.7. Their recommendations ranged between 0.3% for lightly loaded square columns to 1.5% strain for circular columns confined by FRP tubes. While this is a subject matter for further research, 0.6% strain was used for the computation of column drift capacities as recorded during column tests. The computed drift capacities are indicated as vertical lines on hysteretic relationships in Fig. 5.4. The correlation between the analytically computed and experimentally observed drift capacities appear to be good. Therefore, the displacement based design procedure developed by Saatcioglu and Razvi (2002) has been adopted here for confinement of FRP reinforced concrete columns with some limitations, as specified below:

$$A_{Fh} = 14sh_c \frac{f'_c}{f_{Fh}} \left[\frac{A_g}{A_c} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P}{P_o} \delta \quad (5.10)$$

Where;

$$\frac{P}{P_o} \geq 0.2 \quad (5.11)$$

A_{Fh} : Area of total transverse FRP reinforcement parallel to the direction of lateral load.

E_F : Elastic modulus of transverse FRP reinforcement.

f_{Fh} : Maximum stress developed in transverse FRP reinforcement prior to column failure (defined as more than 20% strength decay).

For $P \leq 0.2 P_{ro}$ $f_{Fh} = 0.004 E_F$; and for $P \geq 0.3$ $f_{Fh} = 0.006 E_F$ with linear interpolation for in-between values.

P_{ro} : Concentric compression resistance of FRP reinforced concrete column computed using material resistance factors. Ignoring the contribution of FRP longitudinal reinforcement, $P_{ro} = \phi_c \alpha_l f'_c A_g$

α_l : Rectangular stress block parameter; $\alpha_l = 0.85 - 0.0015 f'_c \geq 0.67$

ϕ_c : Material resistance factor for concrete; $\phi_c = 0.65$.

All other terms in Eq. 5.10 are as described above for Eq. 5.6. The limit specified in Eq. 5.9 also applies.

The design drift capacity, “ δ ” in Eq. 5.10 should not be taken greater than 3%, because of the limited axial deformability of FRP bars in compression. When the ductility related capacity reduction factor $R_d = 2.0$, it is recommended to use $\delta = 0.03$ to provide column deformability associated with collapse prevention performance level for FRP reinforced concrete buildings.

5.5.2.3 Design of FRP Reinforced Concrete Beams for Continuity

Earthquake resistant frames must be designed for continuity and redundancy to maintain structural integrity within the inelastic range of deformations. This translates into a minimum number of continuous beam reinforcement, top and bottom, as well as minimum amounts of top and bottom reinforcement to provide sufficient negative and positive moment resistances throughout the length. The practice that is followed for steel-reinforced concrete beams may also be used for beams reinforced with FRP bars. In view of the brittle nature of FRP reinforcement, it is recommended that the practice used for fully ductile steel reinforced beams be adopted, though the level of ductility expected from the FRP reinforced beams can be “moderate” at best. This implies that negative moment regions near column faces must be designed for positive moments at least equal to 50% of the largest negative moment at either beam end. All other locations along the

length of the beam must be designed to have a minimum positive and negative moment resistance of at least 25% of the highest negative moment at either end. Furthermore, at least $\frac{1}{4}$ of the maximum number of top and bottom bars, but not less than two each must be continuous. These requirements ensure continuity and minimum flexural resistances against unexpected negative and positive moments that may be induced under reversed cyclic loading caused by earthquakes anywhere in the beam.

5.5.2.4 Confinement Reinforcement for FRP Reinforced Concrete Beams

Tests of FRP reinforced concrete beams under reversed cyclic loading indicated that the confinement reinforcement had limited effectiveness in beams because of lack of axial compression and resulting reduction in concrete compression zone (Sharbatdar and Saatcioglu (2007c)). Nevertheless, in over-reinforced elements, which is recommended in earthquake resistant elements, it is a good practice to confine the compression concrete. This reinforcement also restrains the compression reinforcement against buckling. Therefore, it is recommended that the confinement reinforcement requirements presented in Sec. 5.5.2.2 be followed for beam confinement reinforcement, with minimum values specified in Eqs. 5.9 and 5.11 substituted in Eq. 5.10.

5.5.2.5 Design of Beam-Column Connections using FRP Reinforcement

Beam-column joints of reinforced concrete frames are subjected to shear forces during earthquakes. Tension beam reinforcement anchored into the adjoining column applies a pull force on the joint while the compression zone of the same beam applies a push force, creating joint shear. Depending on the level of confinement provided by the beams framing into the joint, the joint concrete can develop certain minimum shear resistance. If this value is exceeded during an earthquake, diagonal shear failure may occur. Therefore, it is a common practice to continue the column confinement reinforcement into the joint either with the same spacing (for joints that are partially enclosed by framing beam elements) or with one half the spacing (for joints that are confined by four framing beams) to control diagonal cracking. This requirement may be adopted from CSA A23.4-2004, and can be employed until further research data on FRP reinforced concrete joints become available.

5.5.2.6 Strength Distribution at Beam-Column Connections

Steel reinforced concrete frame buildings are designed to dissipate seismic induced energy through significant yielding of flexure dominant beams rather than compression dominant columns. Columns, as vertical load carrying elements, are responsible from overall strength and stability of buildings and hence need to be protected against excessive inelastic deformations. This design philosophy is implemented by ensuring higher column capacities at all beam-column joints than the capacities of framing beams. FRP reinforced concrete beams tested by Sharbatdar and Saatcioglu (2007c) indicate that these beams essentially remain elastic, though experience stiffness degradation due to concrete cracking. Therefore, energy dissipation through inelastic deformations of beams may not be possible. However, FRP reinforced concrete elements experience more reduction in stiffness beyond cracking than steel reinforced concrete elements, because of lower elastic modulus of FRP bars. It was concluded in Chapter 3 that effective elastic rigidities of FRP reinforced concrete columns and beams can be taken as 50% and 20% of their gross sectional rigidities (as opposed to 70% and 35% for steel reinforced concrete elements). This indicates that FRP reinforced beams deform significantly within the elastic range, while accommodating column deformations. Therefore, it is not expected that the beam deformation demands will be high enough to result in inelastic action as the framing columns experience limited inelasticity for which they are designed. However, it may be a good practice to safeguard against the use of very deep and strong beams, relative to the adjoining columns, by enforcing a strength limitation at beam-column joints. Accordingly, it is recommended to design columns such that the summation of nominal column moment capacities $(M_n)_{col}$ at a given joint is at least equal to the summation of nominal beam moment capacities $(M_n)_{bm}$ at the joint. This is summarized in Eq. 5.12.

$$\sum (M_n)_{col} \geq \sum (M_n)_{bm} \quad (5.12)$$

Where;

$\sum (M_n)_{col}$: Summation of nominal column moment capacities $\phi_c = \phi_{FRP} = 1.0$ above (if any) and below the joint, each computed under factored axial load.

$\sum (M_n)_{bm}$: Summation of nominal beam moment capacities $\phi_c = \phi_{FRP} = 1.0$ to the left and right of the joint under consideration.

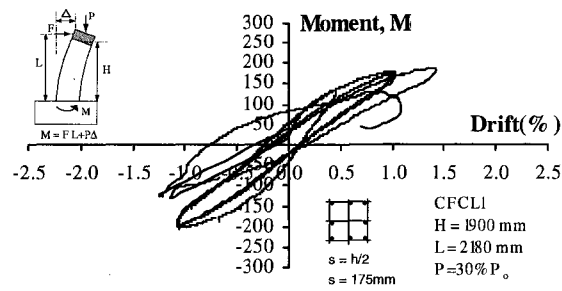
5.5.2.7 Design of FRP Reinforced Concrete Frame Elements for Shear

Shear failure caused by diagonal tension or diagonal compression is a sudden and brittle failure. Therefore, earthquake resistant structural elements are designed to promote flexural response, while shear distress is prevented. This is ensured by providing shear capacity that is higher than the shear force associated with flexural failure. For fully ductile steel reinforced concrete elements subjected to strong earthquake excitation, the flexural resistance may be increased due to the strain hardening of steel reinforcement. Therefore, in design, the shear capacity is provided to be higher than that associated with the development of steel strain hardening. The moment resistance at this increased level is referred to as “Probable Moment Resistance (M_{pr}).” The probable moment resistance is computed by assuming the steel to be stressed to 1.25 times the specified design yield strength, with material resistance factors $\phi_c = \phi_s = 1.0$. This results in approximately a 25% increase in flexural moment resistance with a corresponding increase in shear demand. Shear design force is then computed as the higher of shear force associated with the development of probable moment resistance at member ends or the shear force obtained from seismic analysis, whichever is higher. A similar approach is used for moderately ductile members, except in this case the moment value used for the computation of shear design force is the nominal moment capacity with material resistance factors $\phi_c = \phi_s = 1.0$. Although the strain hardening does not apply to FRP reinforcement, and Section 5.5.1.2 suggests designing FRP reinforced concrete buildings for moderate ductility, any unaccounted increase in seismic demand can translate into an increase in flexural resistance because of elastic behaviour of FRP reinforcement. Therefore, it is prudent to consider this possible increase in shear force level. Until further research becomes available on the topic, it is recommended that FRP reinforced concrete beams and columns are designed similar to fully ductile steel reinforced concrete elements. This translates into a shear design force value equal to the higher of the shear force associated with the development of 1.25 times the nominal sectional capacity at member ends or the applied factored seismic shear forces.

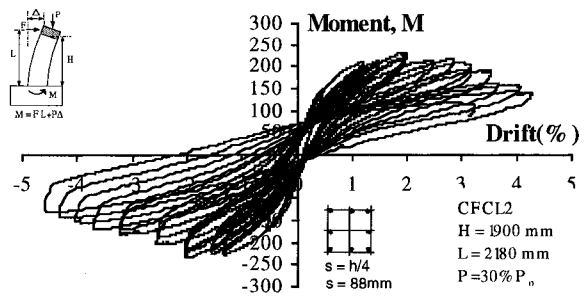
Table 5.1 Tests of FRP Reinforced Concrete Columns under Reversed Cyclic Loading

Specimen	Reinforcement Arrangement	ρ (%)	s (mm)	H (mm)	P/Po (%)	Drift (%)
CFCL1	8-9.5 mm bar	0.27	175	1900	30	1
CFCL2	8-9.5 mm bar	0.55	88	1900	30	3
CFCL3	12-9.5 mm bar	0.37	175	1900	27	2
CFCL4	12-9.5 mm bar	0.73	88	1900	27	3
CFCL6	8-9.5 mm bar	0.55	88	1000	33	2
CFCL9	12-9.5 mm bar	0.73	88	1000	30	2

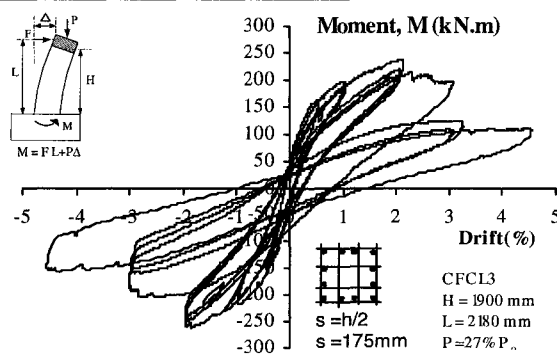
- Notes:**
- 1) All columns have $f'_c = 38$ MPa concrete
 - 2) $\rho = A_{FRP}/b_c s$ where A_{FRP} is the area of FRP grid in a column sectional direction and b_c is the core dimension perpendicular to the transverse FRP reinforcement
 - 3) Drift capacities are approximate rounded values at 20% strength decay
 - 4) All columns were observed to show yield-like behaviour at about 1% drift, with a significant change in the rate of deformations



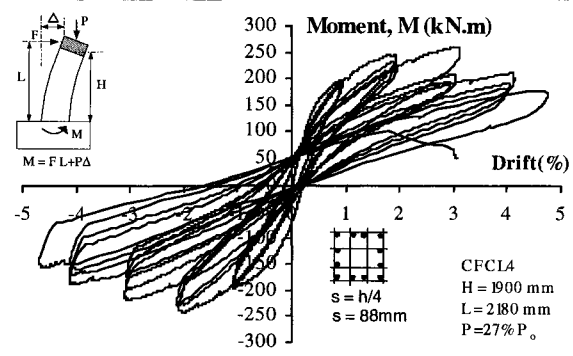
(a) Column CFCL1



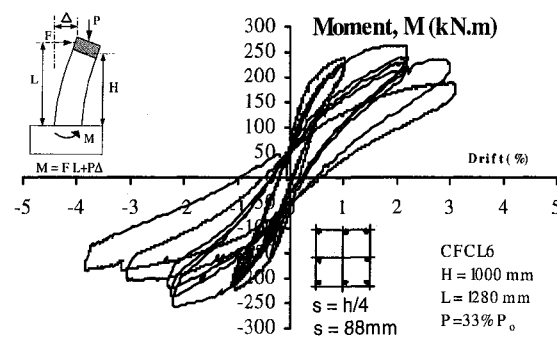
(b) Column CFCL2



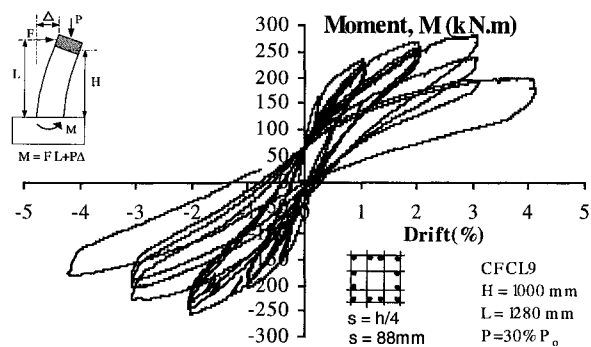
(c) Column CFCL3



(d) Column CFCL4

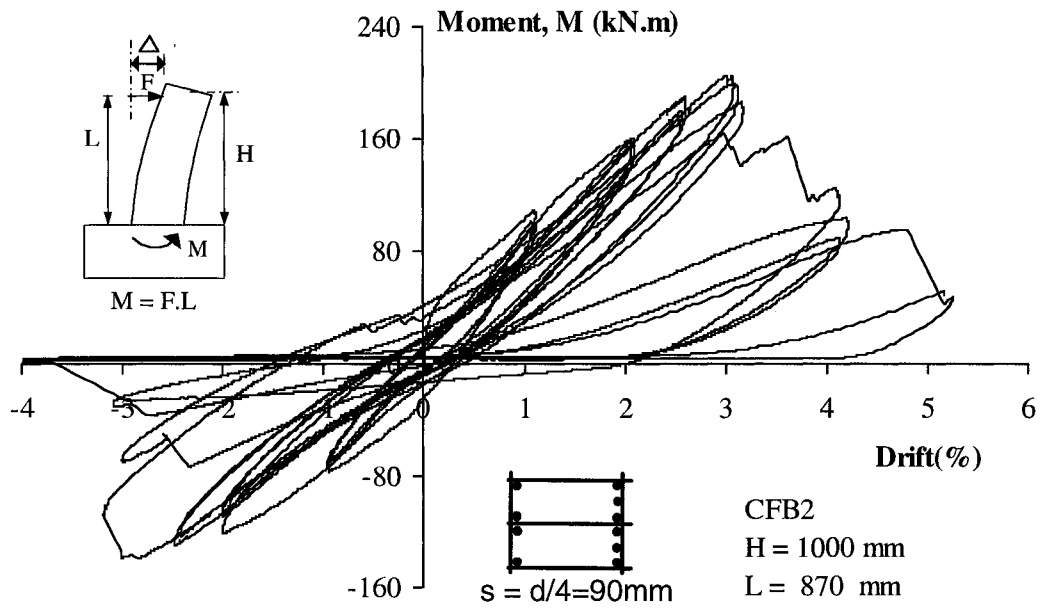


(e) Column CFCL6

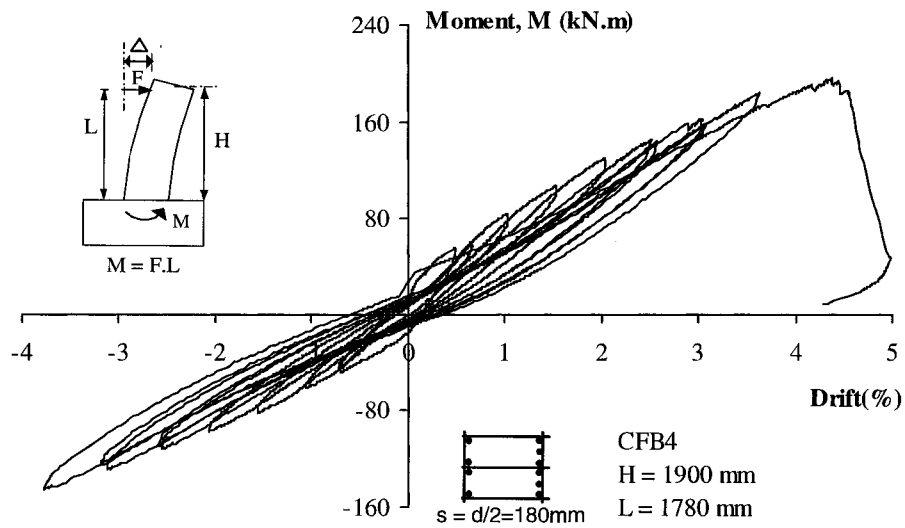


(f) Column CFCL9

Figure 5.1 Hysteretic behaviour of columns reinforced with CFRP bars and confined with CFRP Grids

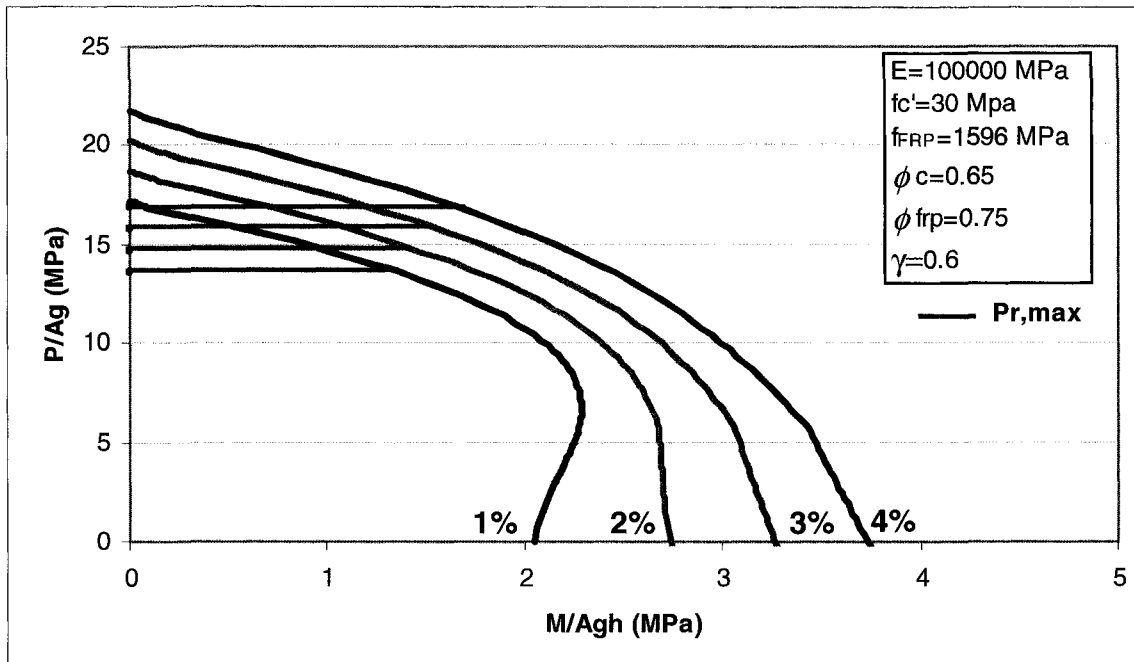


a) Beam CFB2 with 1000 mm Shear Span

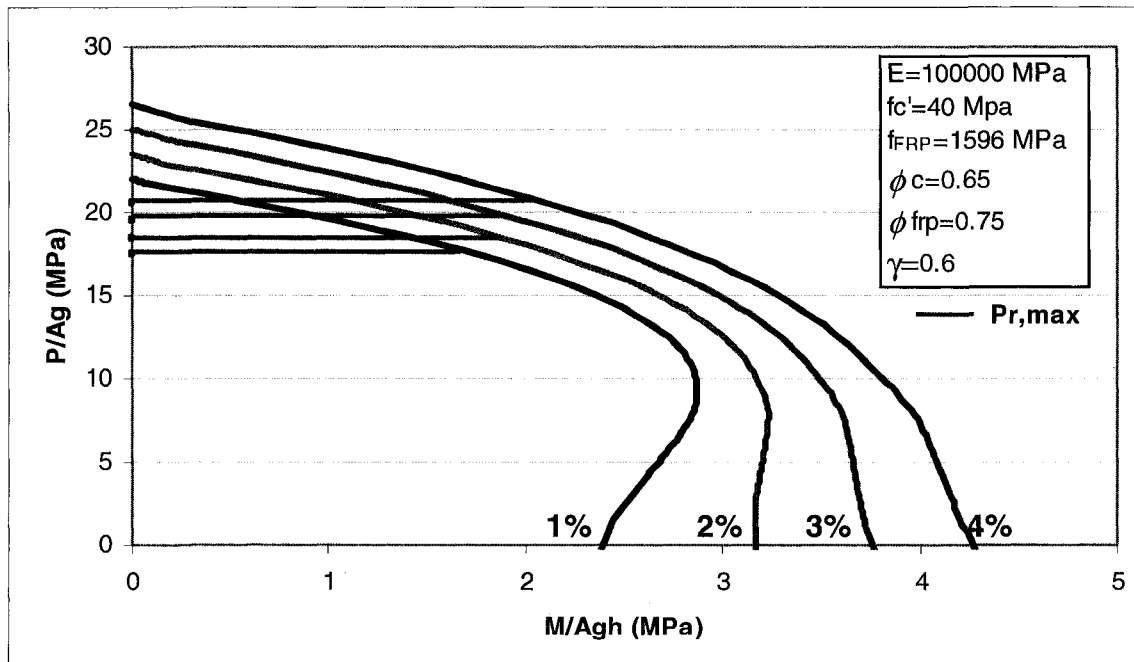


b) Beam CFB4 with 1900 mm Shear Span

Figure 5.2 Hysteretic relationships for FRP reinforced concrete beams under reversed cyclic loading

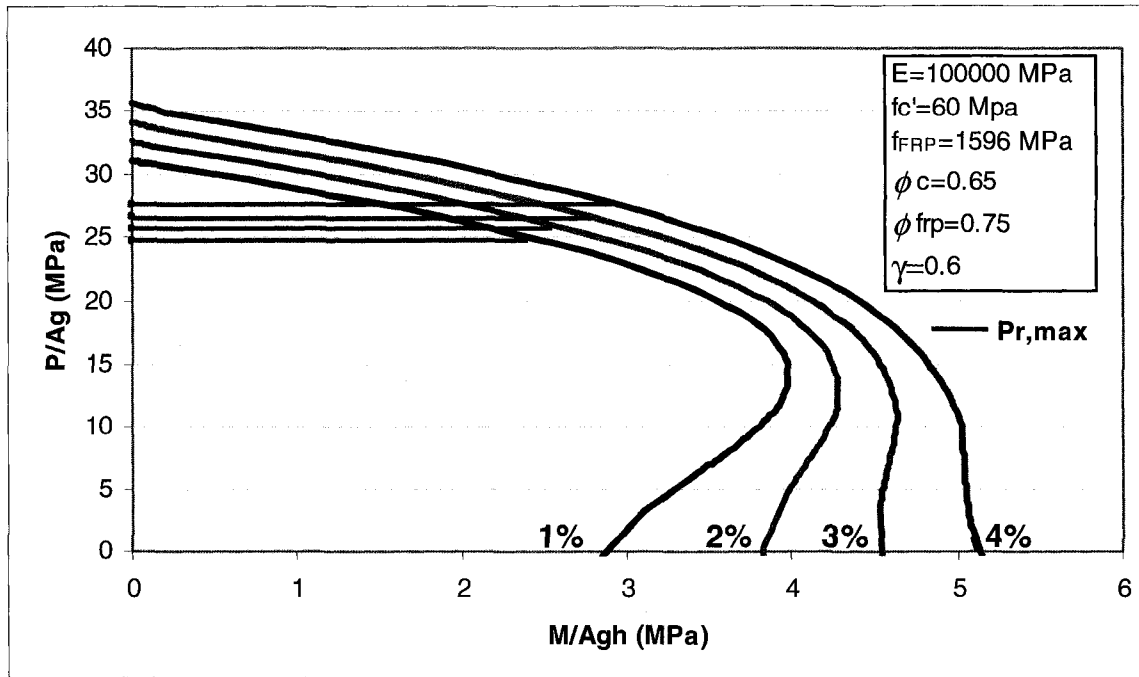


(a)

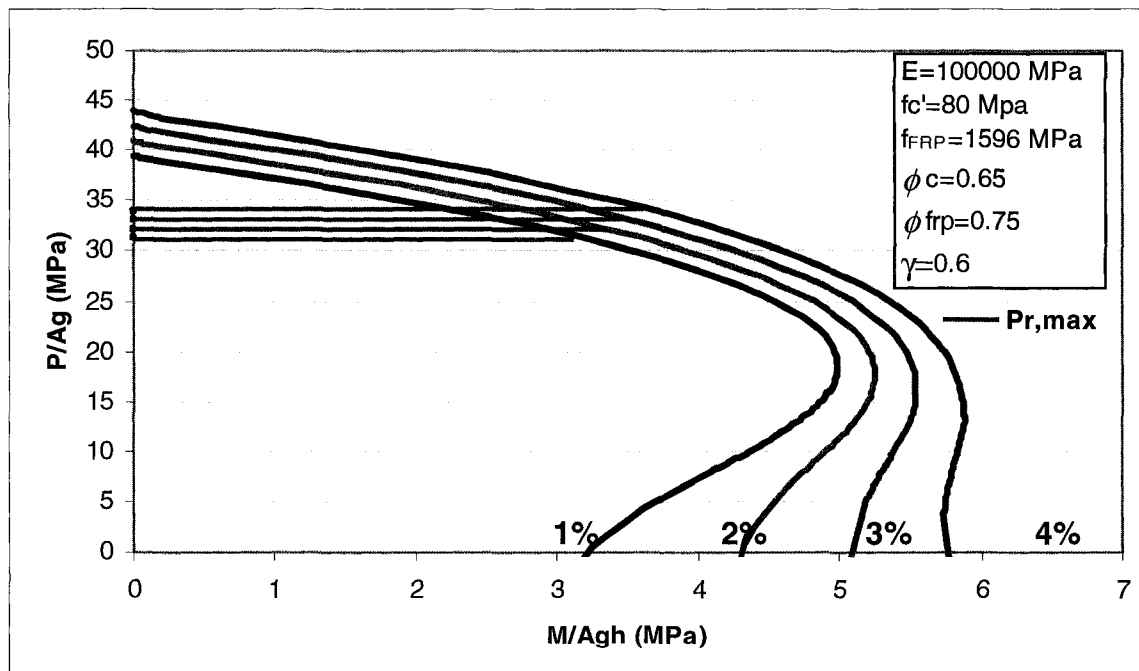


(b)

Figure 5.3 Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

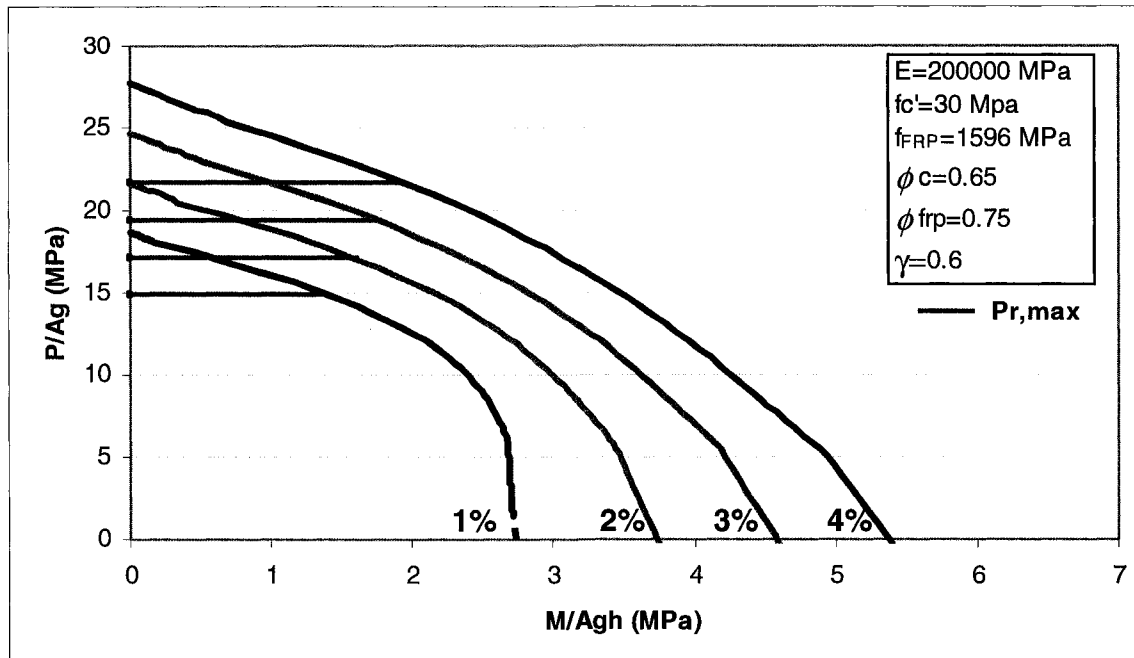


(c)

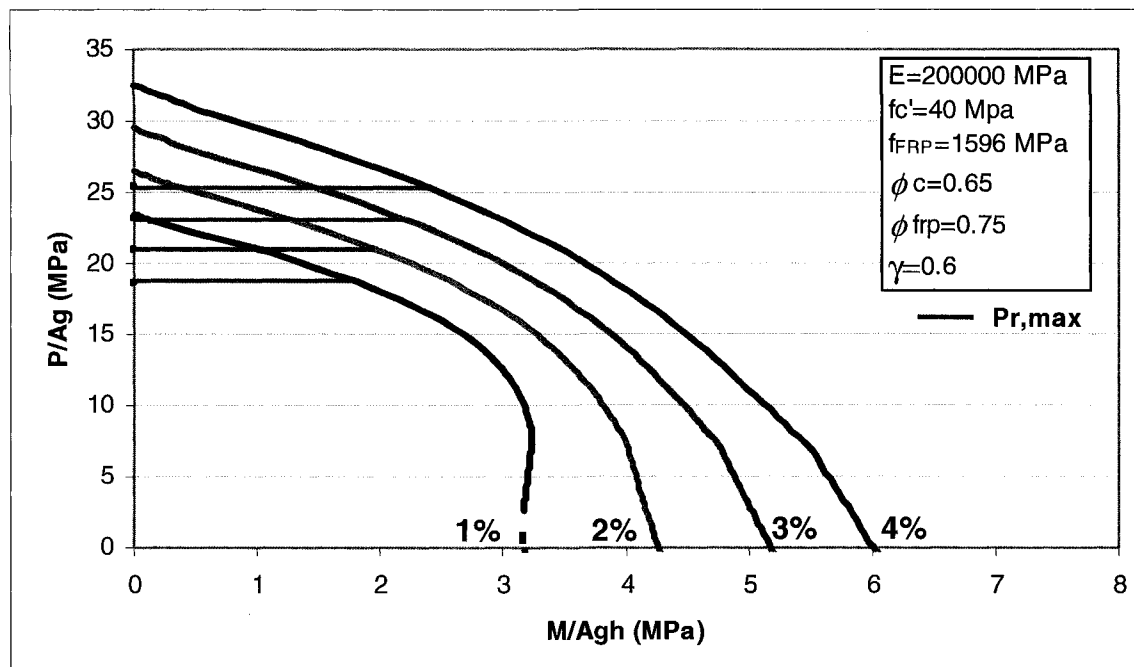


(d)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

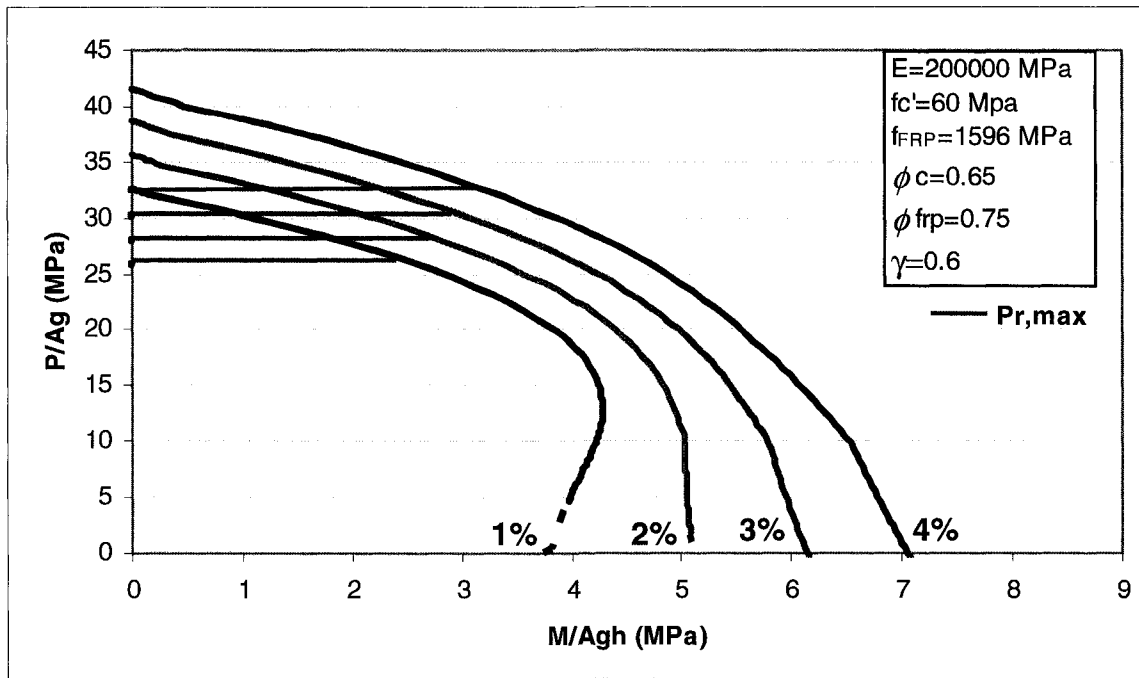


(e)

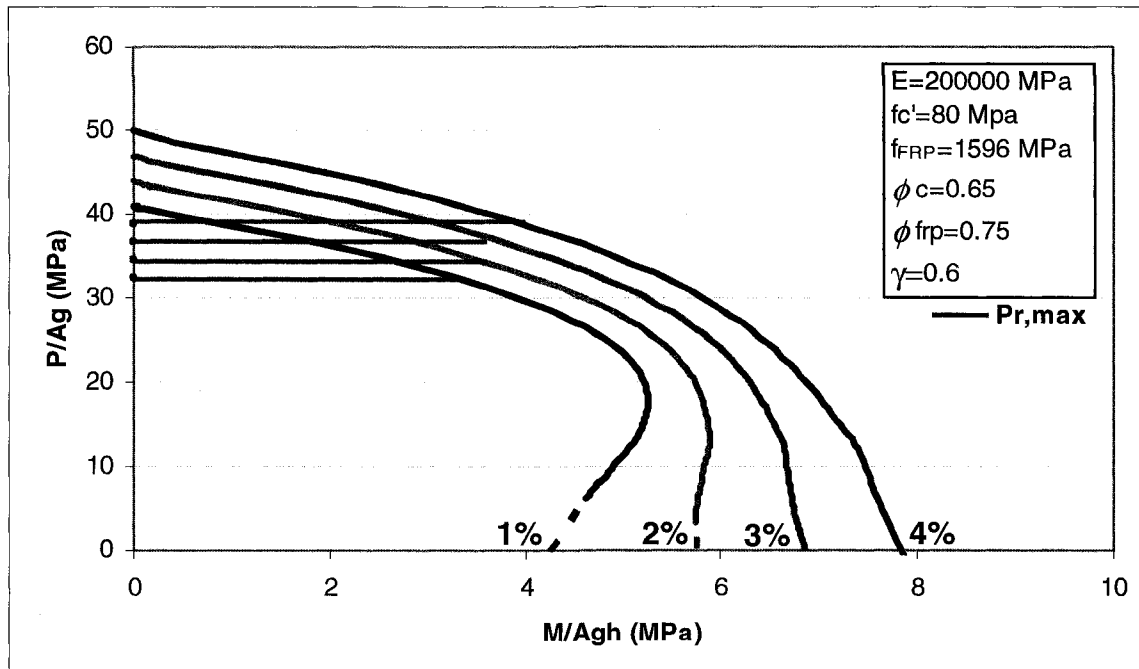


(f)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

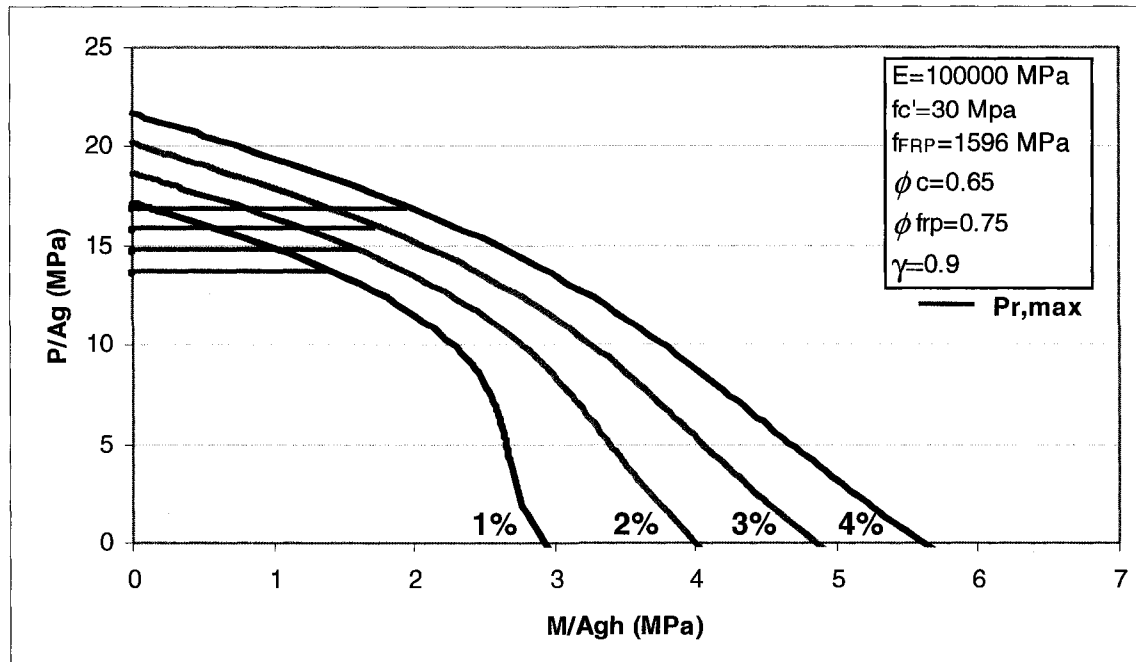


(g)

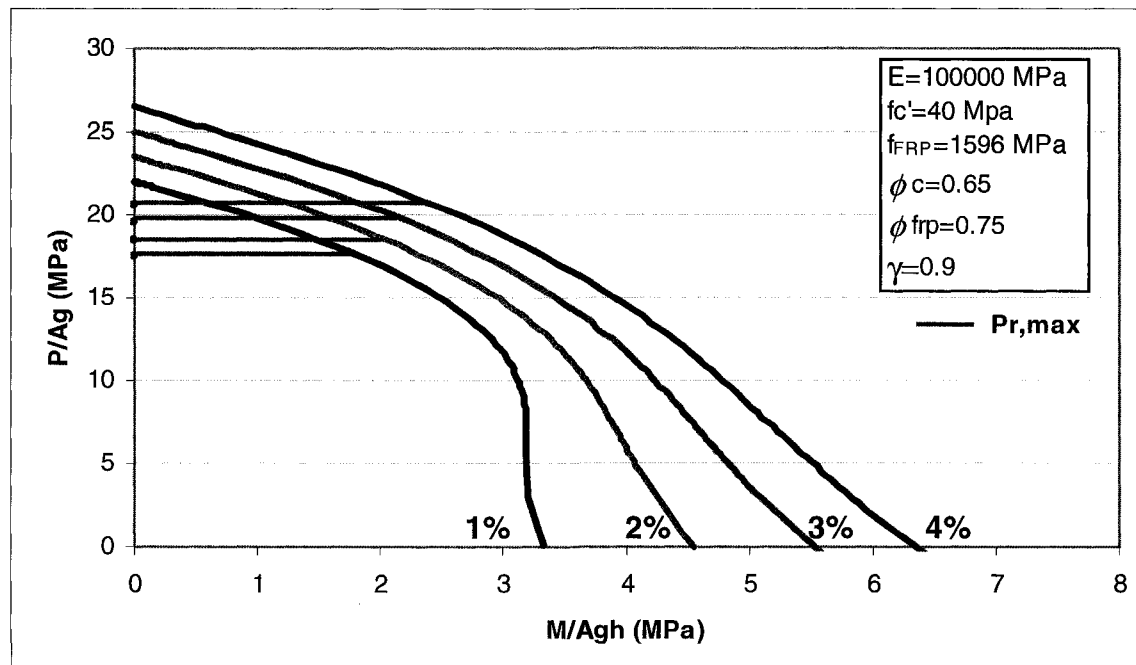


(h)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

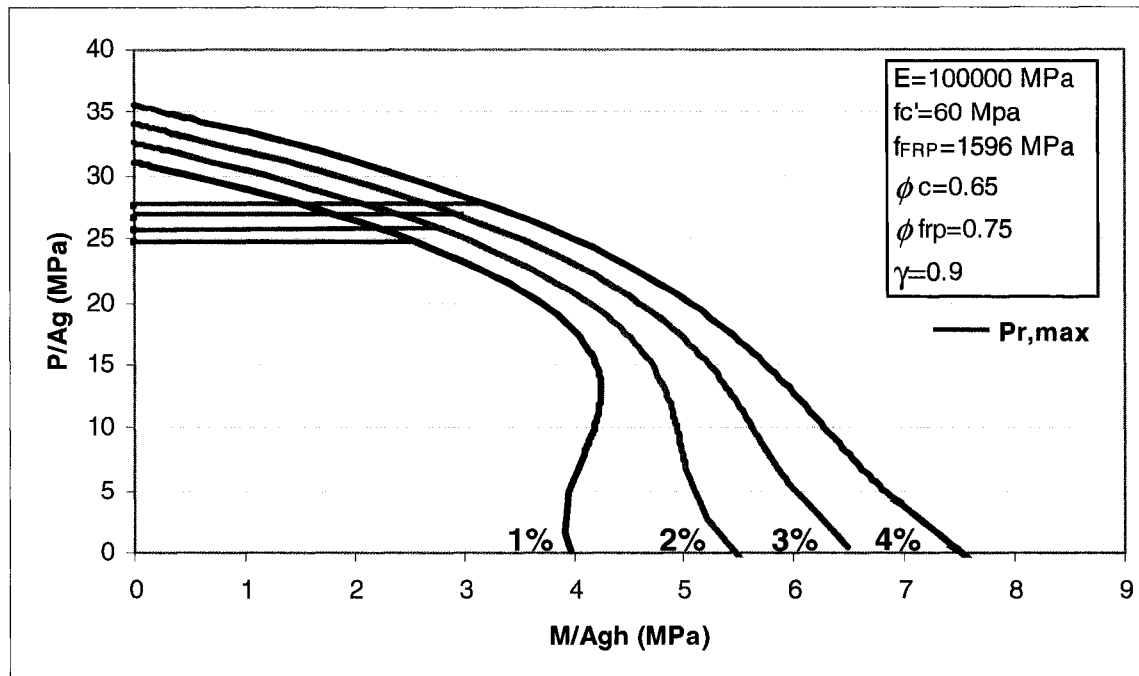


(i)

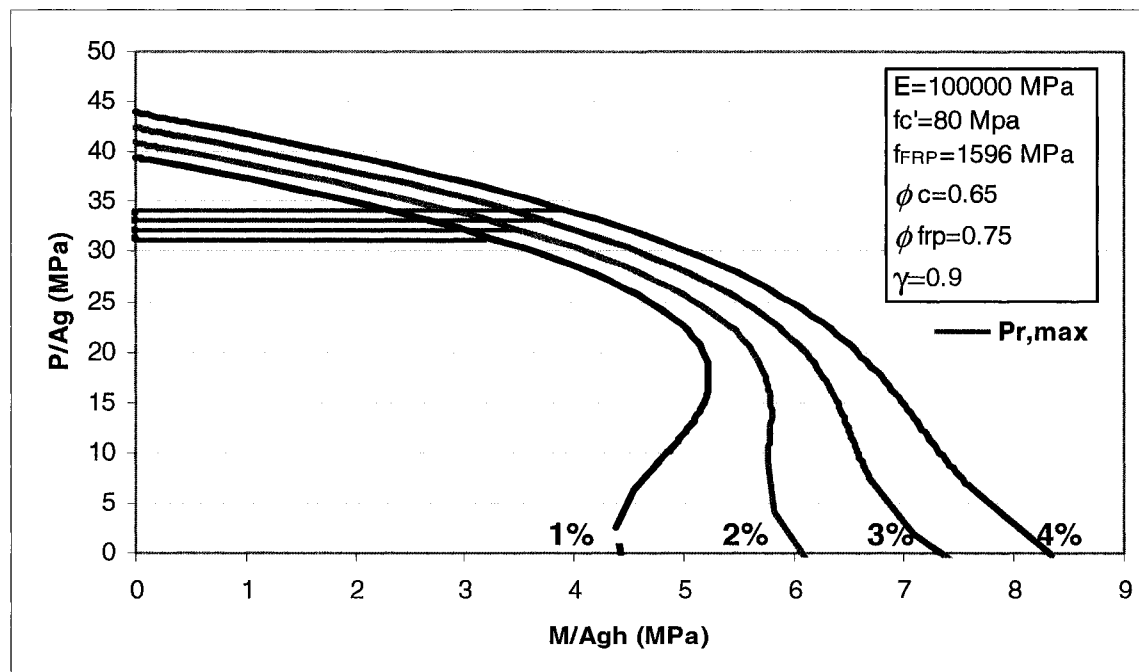


(j)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

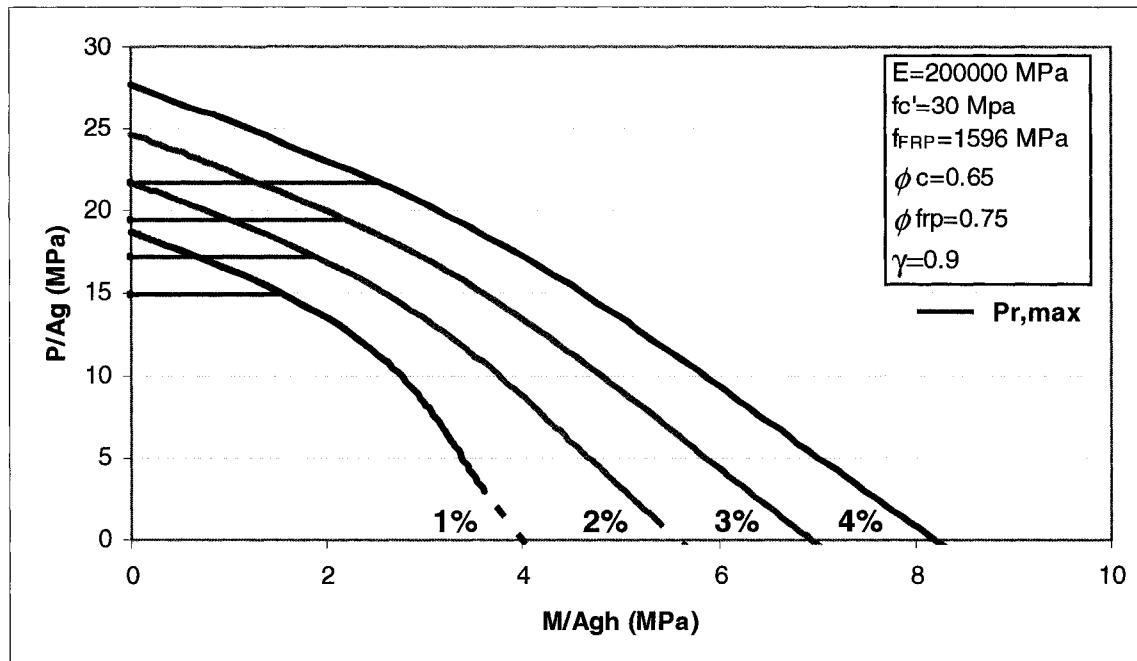


(k)

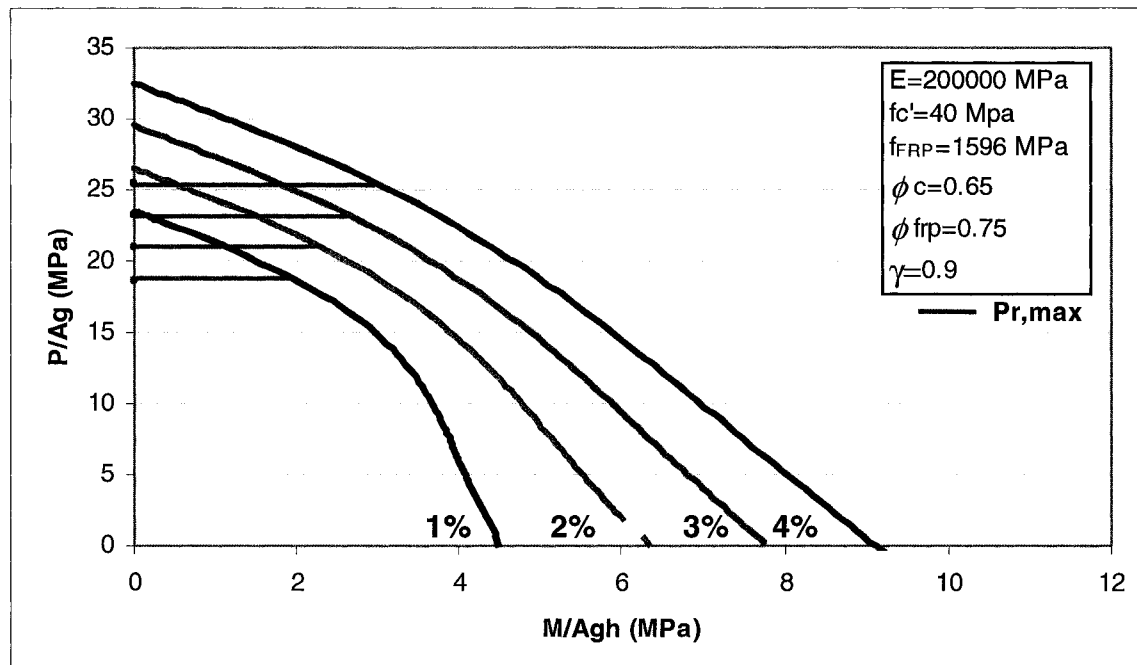


(l)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

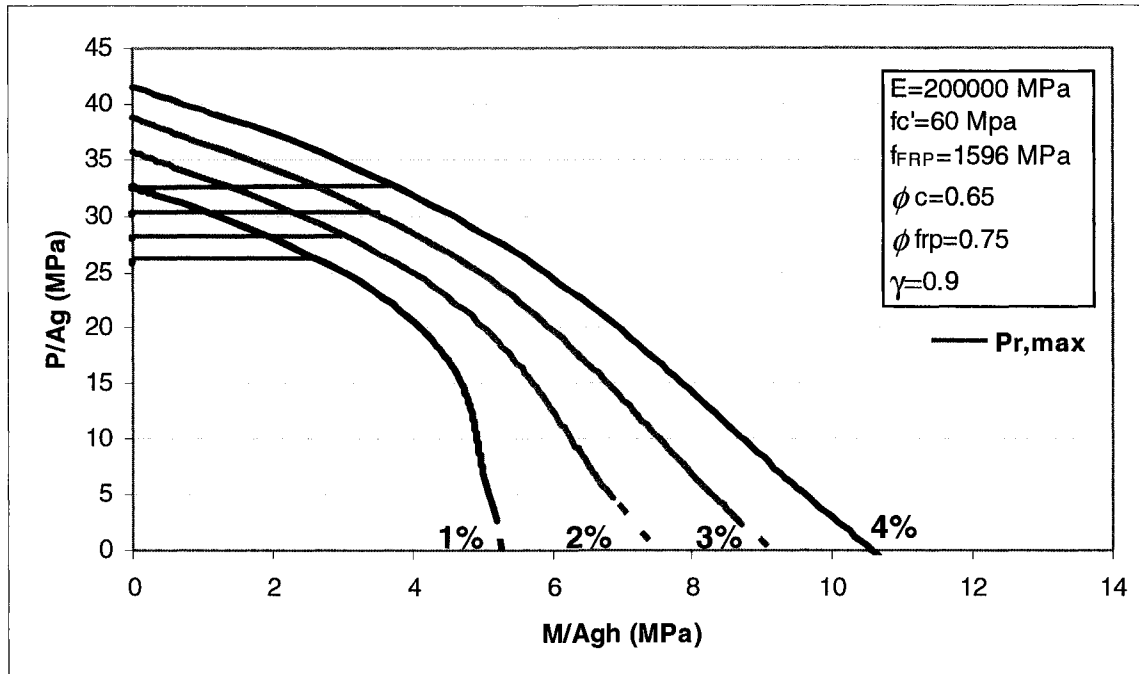


(m)

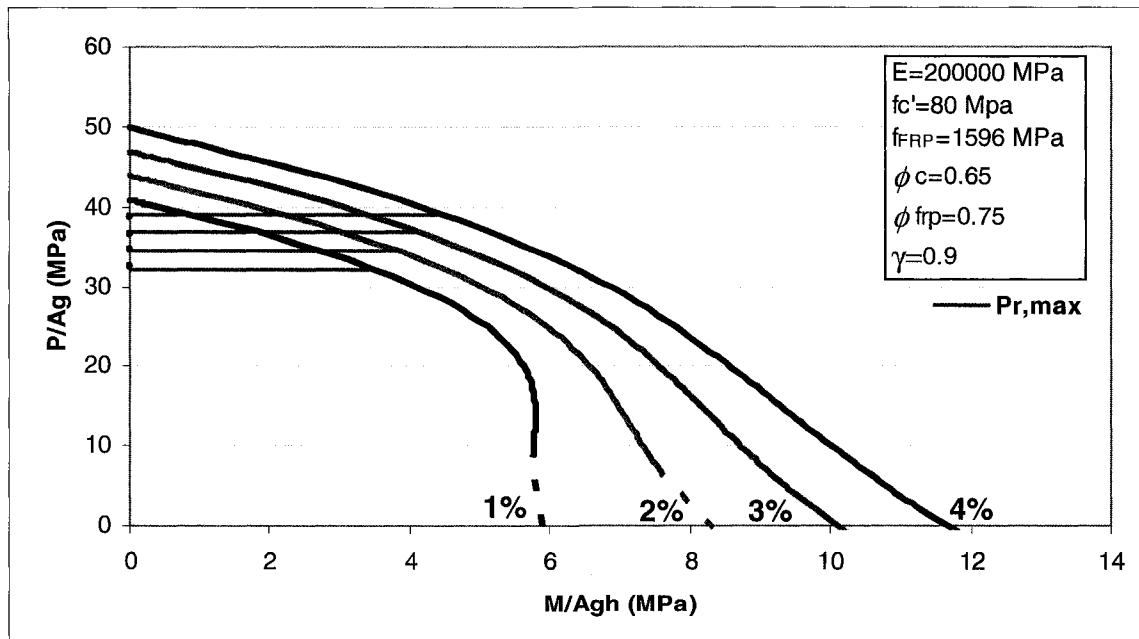


(n)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns



(o)



(p)

Figure 5.3 (Cont'd) Moment-Axial Force Interaction Diagrams for FRP Reinforced Concrete Columns

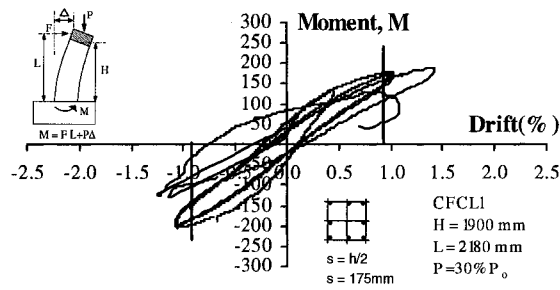


a) CFRP bar failure in compression

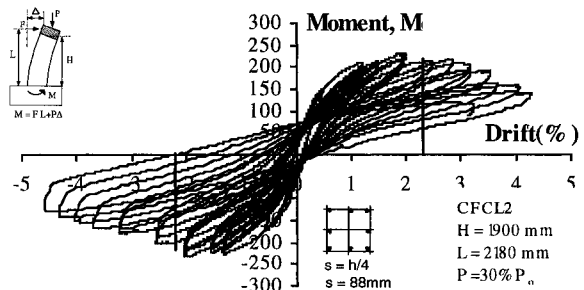


b) Steel bar buckling in compression

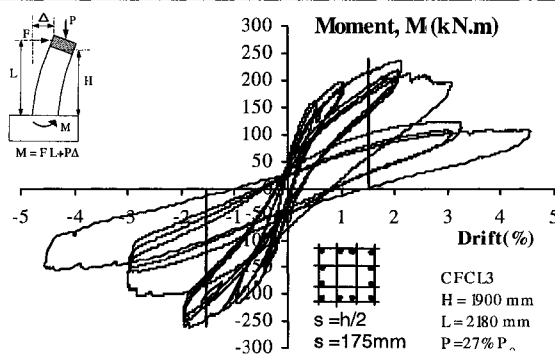
Figure 5.4 Compression failures of FRP and steel bars in columns



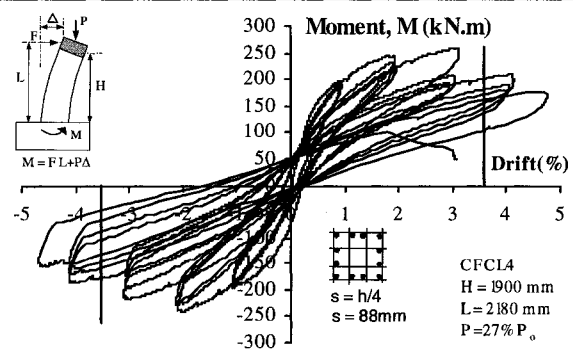
(a) Column CFCL1



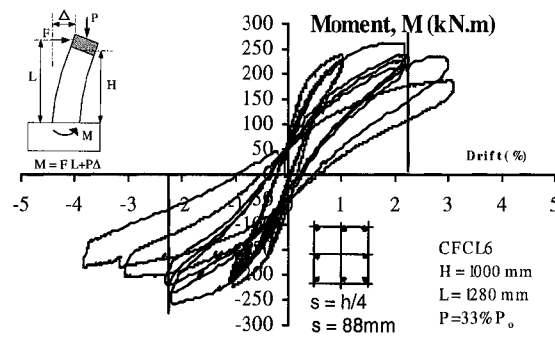
(b) Column CFCL2



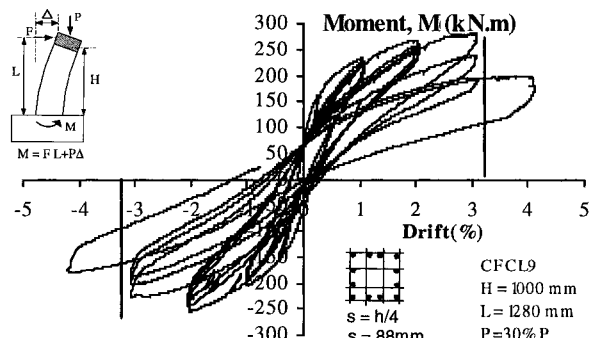
(c) Column CFCL3



(d) Column CFCL4



(e) Column CFCL6



(f) Column CFCL9

Figure 5.5 Computed drift ratios (indicated by vertical lines)

Chapter 6

Summary and Conclusions

6.1 Summary

The use of fibre reinforced polymer (FRP) reinforcement in buildings is gaining acceptance in the construction industry because of its corrosion resistance, durability and higher strength. However, research on seismic performance of FRP reinforced concrete structures is currently limited worldwide. The current codes and standards, including the CSA S608-02 (2002), ISIS Manual (2001) and ACI 440 (2006) limit the use of FRP reinforcement in structural concrete, especially for earthquake resistant construction. Recognizing this deficiency, the current research project was undertaken to develop earthquake resistant design information and seismic design guidelines for FRP reinforced concrete buildings in Canada.

One of the first tasks of the research program was to develop a computer program (SEQUAKE) for planar static and dynamic inelastic response history analysis of FRP and steel reinforced concrete structures. This tool was essential to be able to conduct dynamic analysis, through which seismic force and deformation demands were established. The solution of equation motion was done by using the step-by-step integration (Newmark constant acceleration) method. This numerical technique allowed the treatment inelastic deformations, with continuously changing element stiffnesses. Plastic deformations of elements are assumed to be concentrated in plastic hinges provided at member end springs.

The moment-rotation relationships of steel and FRP reinforced concrete members are obtained by incorporating hysteretic models of Clough (1966) and Shahbatdar and Saatcioglu (2003), respectively. The rigid body rotation of the member due to the

anchorage slip of flexural reinforcement was also accounted for through percentage reduction in flexural stiffness.

A total of 12 concrete buildings, reinforced with CFRP and steel rebars, were selected in order to conduct a comparative study. Each set of buildings was designed for Ottawa and Vancouver, representing medium and high seismicity regions in Canada. The buildings consisted of 5, 10 and 15 storey-frames as representatives of low-rise, medium-rise and high-rise buildings. The design of the CFRP reinforced structures were carried out following the requirements of CSA Standard S806 (2002) and ISIS (2001) Manual, while the steel reinforced concrete building was designed using the CSA Standard A23.3 (2004). The design base shears were calculated according to the equivalent static load procedure of NBCC 2005.

Once the buildings were designed, they were analyzed under selected ground motion records. Two different types of UHS compatible ground motion records were selected; i) those for Vancouver and ii) those for Ottawa. In addition, the Vancouver buildings were also analyzed with previously recorded earthquake motions. Computer software SEQUAKE, developed as part of the current investigation, was used to conduct inelastic time history analysis. The results were evaluated in terms of base shear, drift ratios, moment-chord rotation relationships, effects of anchorage slip and nonlinear response.

The analysis results provided seismic force and deformation demands for buildings designed for two representative cities in Canada, i.e., Vancouver and Ottawa with high and medium seismicity reflecting western and eastern seismicity, respectively. The current performance-levels accepted by FEMA 356 for steel reinforced concrete buildings were reviewed. Test results on FRP reinforced concrete columns and beams were evaluated to establish the inelastic deformation capacities of FRP reinforced concrete elements under reversed cyclic loading. This information was used, along with the existing design guidelines for steel reinforced concrete buildings to develop seismic design guidelines for FRP reinforced concrete buildings in Canada. A set of column

interaction diagrams was generated for square FRP reinforced concrete columns as designed aids.

6.2 Conclusions

The following conclusions can be drawn from the analytical investigation presented in this thesis:

- The computer program SEQUAKE, developed as part of the current investigation, can be used as an effective computational tool for dynamic inelastic response history analysis of FRP reinforced concrete structures.
- Seismic base shear demands computed through dynamic response history analyses were significantly lower than the design base shear forces calculated through equivalent static load analysis. This was attributed, in major part, to differences in period calculations based on the empirical code expression and those obtained from dynamic analysis. It was also concluded that the difference between design response spectra in NBCC 2005 and the spectra established for compatible records could lead to additional differences in design force levels.
- The effect of anchorage slip may cause appreciable differences in seismic force demands. Anchorage slip effects elongate the fundamental period. Depending on the spectral shape for a given seismic record it may reduce or increase the demands. It was concluded that anchorage slip should be considered in establishing effective elastic stiffnesses of members as well as the fundamental period. A set of reduction factors for different levels of axial loads was provided to take into account stiffness softening caused by anchorage slip.
- The analyses under intensified earthquake records showed that it is possible to attain inelasticity in FRP reinforced concrete buildings with reductions in seismic force demands relative to elastically computed force levels.
- The drift demands for CFRP reinforced concrete buildings both in Ottawa and Vancouver were comparable to those obtained for steel reinforced concrete buildings implying that similar performance levels can be attained during moderate to strong earthquakes.

- The seismic design forces specified in NBCC 2005 for steel reinforced concrete frame buildings were shown to be applicable for CFRP reinforced concrete buildings, with appropriate modifications. Accordingly, it was recommended that;
 - Effective elastic stiffnesses to be used in the calculation of fundamental periods of CFRP reinforced concrete structures, as well as modeling for dynamic analysis can be taken as $0.2EI_g$ and $0.5EI_g$ for beams and columns, respectively.
 - Ductility related force modification factor R_d should be taken as 1.0 for all CFRP reinforced concrete buildings, except for frame buildings with columns designed to confirm the confinement requirements of Section 5.3.2, in which case it can be taken as 2.0.
 - Over-strength related force modification factor R_o should be taken as 1.0 for all CFRP reinforced concrete buildings, except for frame buildings with columns designed to confirm the confinement requirements of Section 5.3.2, in which case it can be taken as 1.6.
- The strength of CFRP reinforced concrete elements can be computed using standard plane section analysis. However, the characteristics of CFRP reinforcement should be incorporated into the analysis. The compression strength and modulus of elasticity of CFRP reinforcement in compression can be as low as 20% of their values in tension.
- Concrete confinement plays an important role in the design of CFRP reinforced concrete columns. Concrete columns reinforced with longitudinal CFRP rebars and CFRP transverse grids can develop inelasticity in flexure through the confinement of compression zone. The displacement-based confinement design procedure developed by Saatcioglu and Razvi (2002) can be adapted to FRP reinforced concrete columns for design. The procedure has been shown to correlate well with drift capacities recorded experimentally.
- The design of FRP reinforced concrete beams in earthquake resistant frames should ensure continuity and minimum flexural resistance against unexpected negative and positive moments. This can be ensured by following current design practice used for steel reinforced concrete beams.

- The confinement reinforcement requirements presented for CFRP reinforced columns are also applicable for beams, with minimum axial force value specified for columns. This is a good practice in over-reinforced earthquake resistant elements and also helps to restrain the compression reinforcement against buckling.
- Beam-column joints of FRP reinforced concrete frames can be designed to follow the current procedure used for steel reinforced concrete frames.
- CFRP reinforced beams deform significantly within the elastic range, while accommodating the deformations at the ends of framing columns. Therefore, it is not expected that the beam deformation demands will be high enough to result in inelastic action. However, it was recommended to design columns such that the nominal column moment capacities at a given joint is at least equal to the summation of nominal beam moment capacities at the joint in order to safeguard buildings against the use of very deep and strong beams relative to the adjoining columns.
- It was recommended to design CFRP reinforced columns and beams for shear force values equal to the higher of the shear force associated with the development of 1.25 times the nominal sectional capacity at member ends or the applied factored seismic shear forces, considering the possibility of any unaccounted increase in shear force level, until further research becomes available.

6.3 Original Contributions to the Field of Structural Engineering

The following original contributions were made to the field of structural engineering in the area of FRP reinforced concrete buildings:

- A new computer program (SEQUAKE) was developed for dynamic inelastic response history analysis of FRP reinforced concrete buildings, to the author's knowledge incorporating the only existing hysteretic model for FRP reinforced concrete elements.
- 6 FRP reinforced concrete frame buildings were designed for earthquake effects, demonstrating the feasibility of seismic design.

- Seismic force and deformation demands were established for low-rise, medium-rise and high-rise frame buildings located in two different seismic regions of Canada with two different levels of seismicity.
- A design procedure was developed for earthquake resistant FRP reinforced concrete frame buildings in Canada. The procedure provides most relevant aspects of seismic design, as permitted by the current state of knowledge on behaviour of FRP reinforcement and FRP reinforced concrete structures.
- A displacement based design procedure was developed for the confinement of FRP reinforced concrete elements by adapting an existing procedure previously proposed for steel reinforced concrete elements.
- Column interaction diagrams were generated for FRP reinforced concrete columns for the first time in structural design practice.
- First attempts were made to explore and propose earthquake resistant design procedures for concrete buildings reinforced with FRP bars.

6.4 Recommendations for Future Work

The research work presented in this thesis involves the first attempt towards attaining complete design information for FRP new reinforced concrete structures. Therefore, many aspects of practice need further research. Specifically, the following research projects are recommended as future work;

- Research on establishing inelastic strength and deformation demands of FRP reinforced concrete structural elements and assemblies (including beam-column joints) through experimental research.
- Further experimental research on GFRP (glass fibre reinforced polymer) and AFRP (aramid fibre reinforced polymer) reinforced elements subjected to cyclic loading.
- The use of different FRP reinforcement, including GFRP (glass fibre reinforced polymer) and AFRP (aramid fibre reinforced polymer) in concrete elements of earthquake resistant buildings.

- The extension of the principles of seismic design to other types of building structures, including frame-shear wall interactive systems and shear wall structures.
- Development of design procedures for FRP reinforced beam-column joints, shear walls and coupling beams.
- Development of shear design recommendations for FRP reinforced concrete elements.
- Development of computer software for nonlinear push-over analysis of FRP reinforced concrete buildings.
- Improved hysteretic models for columns and beams incorporating the effects of changing axial loads during response.

APPENDIX 1

Stiffness coefficients for a beam with end springs

The stiffness coefficient k_{33} is the moment required at node i to have unit rotation at the same node, as shown in Fig A1.1.

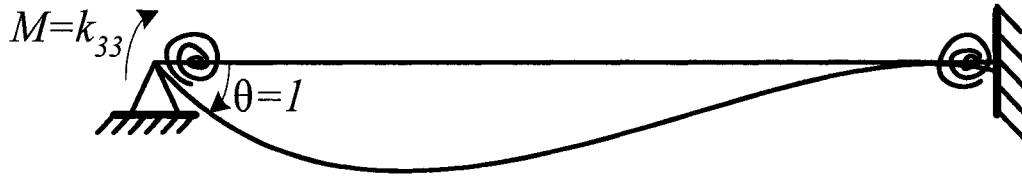


Figure A1.1 Beam with unit rotation at node i due to the moment M and zero rotation at node j due to the fixed support

In order to find the relation between M and θ , the force-method can be used by selecting the statically determinate system as in Fig.A1.2. The moment X is the reaction due to the fixation at node j. The shear deformation effect is included. The axial deformation and the second-order effects are neglected.



Figure A1.2 Beam unit rotation at node i due to the moment M and zero rotation at node j due to the support reaction X

The contribution of end moments M and X to rotation at node j can be expressed by using virtual work method.

$$1.\theta_{a,j} = \int_L \frac{M}{EI} m dx + \int_L \frac{V}{GA_s} v dx \quad (A1.1)$$

$$1.\theta_{b,j} = \int_L \frac{X}{EI} m dx + \int_L \frac{V_x}{GA_s} v dx + \frac{X}{k_{in,eff,j}} \quad (A1.2)$$

Where, $\theta_{a,j}$ is the rotation at node j due to the moment M at node i, $\theta_{b,j}$ is the rotation at node j due to the moment X at node j, m is the unit moment at node j. First term at the right hand side of the Eq.A1.1 is the deformation due to flexure and the second term is the deformation due to shear. In equations, A_s is the shear-reduced cross section of the beam. Since there is fixation at node j, the total rotation vanishes, i.e., $\theta_a + \theta_b = 0$. From the above equations, X can be expressed in terms of M as;

$$X = - \frac{\frac{1}{6} \frac{L}{EI} - \frac{1}{GA_s L}}{\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}}} M \quad (A1.3)$$

By applying the same principle to obtain the rotation at node i, the contributions of the moments M and X can be written as;

$$1.\theta_{a,i} = \int_L \frac{M}{EI} m dx + \int_L \frac{V}{GA_s} v dx + \frac{M}{k_{in,eff,i}} \quad (A1.4)$$

$$1.\theta_{b,i} = \int_L \frac{X}{EI} m dx + \int_L \frac{V_x}{GA_s} v dx \quad (A1.5)$$

Where, $\theta_{a,i}$ is the rotation at node i due to the moment M at node i, $\theta_{b,i}$ is the rotation at node i due to the moment X at node j, m is the unit moment at node i. From Eqs. A1.4 and A1.5, for unit rotation at node i, the stiffness coefficient can be obtained as;

$$k_{33} = \frac{\left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right)}{\left(\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right) \left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right) - \left(\frac{L}{6EI} - \frac{1}{GA_s L} \right)^2} \quad (A1.6)$$

By applying the similar procedures, the k_{66} , k_{36} , k_{63} , k_{23} , k_{56} , k_{53} , k_{26} , k_{32} , k_{35} , k_{65} , k_{62} , k_{22} , k_{25} , k_{52} and k_{55} can be written as;

$$k_{66} = \frac{\left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right)}{\left(\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right) \left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right) - \left(\frac{L}{6EI} - \frac{1}{GA_s L} \right)^2} \quad (A1.7)$$

$$k_{36} = k_{63} = \frac{\frac{L}{6EI} - \frac{1}{GA_s L}}{\left(\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right) \left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right) - \left(\frac{L}{6EI} - \frac{1}{GA_s L} \right)^2} \quad (A1.8)$$

$$k_{23} = \frac{\left(\frac{L}{2EI} + \frac{1}{k_{in,eff,j}} \right)}{\left(\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right) \left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right) - \left(\frac{L}{6EI} - \frac{1}{GA_s L} \right)^2} \quad (A1.9)$$

$$k_{56} = - \frac{\left(\frac{L}{2EI} + \frac{1}{k_{in,eff,i}} \right)}{\left(\frac{1}{3} \frac{L}{EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,i}} \right) \left(\frac{L}{3EI} + \frac{1}{GA_s L} + \frac{1}{k_{in,eff,j}} \right) - \left(\frac{L}{6EI} - \frac{1}{GA_s L} \right)^2} \quad (A1.10)$$

$$k_{53} = -k_{23}, k_{26} = -k_{56}, k_{32} = k_{23}, k_{35} = k_{53}, k_{65} = k_{56}, k_{62} = k_{26} \quad (\text{A1.11 a, b, c, d, e, f})$$

$$k_{22} = \frac{k_{23} + k_{56}}{L}, k_{25} = \frac{k_{35} + k_{65}}{L}, k_{52} = -k_{22}, k_{55} = k_{22} \quad (\text{A1.12 a, b, c, d})$$

When there is no plastification, spring constants are taken as large numbers ($k_{in,eff,i} \rightarrow \infty$ and $k_{in,eff,j} \rightarrow \infty$) so that they do not change the element elastic stiffness values.

In three-dimensional analysis, the stiffness matrix, $[K]_{12 \times 12}$, includes 144 stiffness coefficients. The additional flexural stiffness coefficients in the perpendicular direction can be determined in a similar way. Torsional stiffness coefficient is:

$$k_{mn} = \frac{GI_x}{L} \quad (\text{A1.13})$$

APPENDIX 2

SEQUAKE User Guide

SEQUAKE is a static and dynamic analysis program for the reinforced concrete structures. The program features includes:

- Static analysis
- Eigen value analysis
- Inelastic dynamic analysis of steel and FRP reinforces structures

In the program the assemblage procedures used for all methods are adopted from an open source program developed by Erkmen (2001).

The program is developed for the three-dimensional analyses of structures. Therefore, all data entries are arranged for three-dimensional structures. However, hysteretic behaviour of columns does not include the effects of biaxial bending. Therefore, in this study, all structures are selected as planar frames and the analyses are conducted in one plane.

It should also be noted that due to the definition of the hysteretic model the change in the axial forces of the members are not taken into account in the calculation of stiffness coefficients. As shown in Appendix I, the shear deformations are included in the elastic stiffness coefficients; however inelastic shear effects are omitted. Mass of the floors can be distributed to multiple nodes provided that those nodes are defined as master nodes. The effects of static loads are included in the dynamic analysis. However it assumed that the static loads alone do not cause any nonlinear behavior. Therefore, the program applies all of the static loads at once and not gradually.

A2.1. Global and local coordinate systems

Global and local coordinate systems are shown in Fig. A2.1.

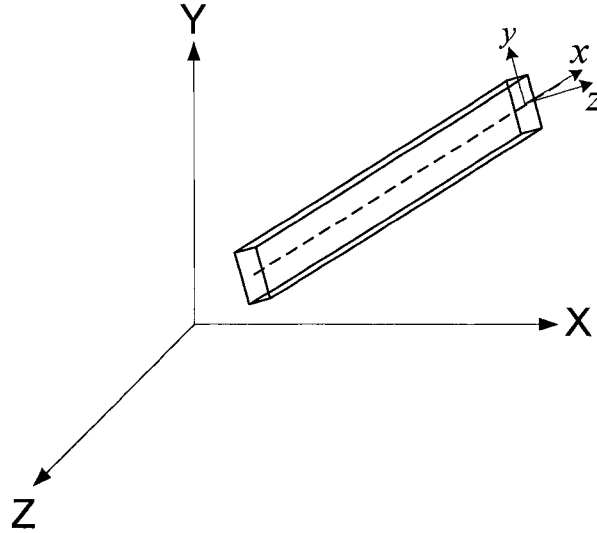


Figure A2.1 Global and local coordinate systems

The direction of local y axis is defined with the angle β , which is the angle between the local y axis and the plane containing both local x and global Y axes as shown in figure A2.2, (Erkmen 2001). The angle β is always between 0^0 and 90^0 . Some examples of angle β , are shown in Fig. A2.3.

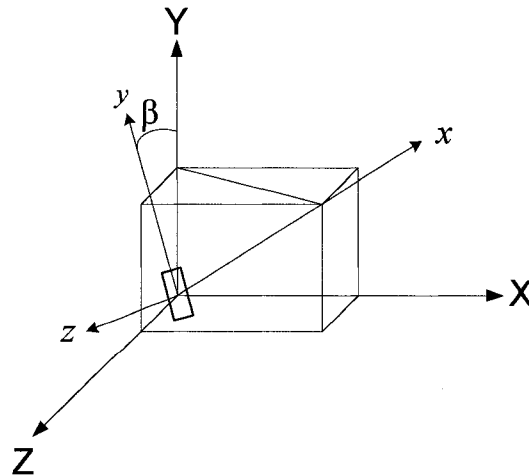


Figure A2.2 Definition of angle β

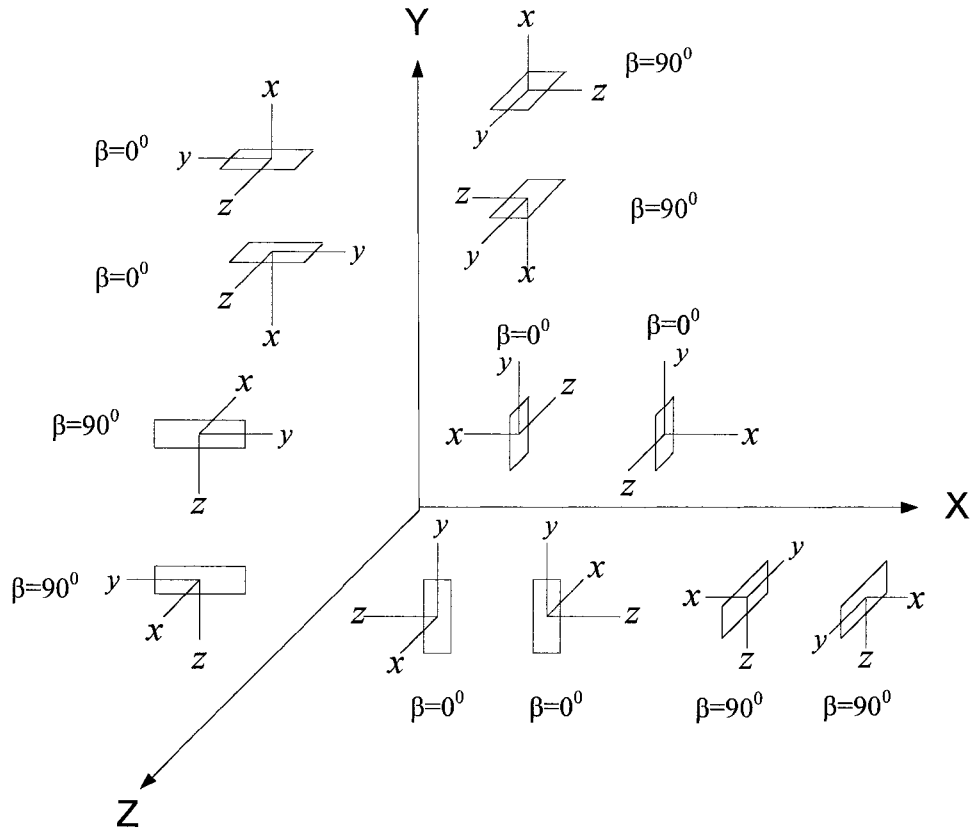


Figure A2.3 Examples of angle β

A2.2. Data entry and solutions

Program accepts a group of input data files with .TXT extension and creates another group of output files with .SEQ extension.

A2.2.1 Input files for static and eigen value analyses

The input files required for the static and eigen value analyses are:

- GEO.TXT
- DIAPHRAGM.TXT
- LOADS.TXT
- MASS.TXT
- PROPERTY.TXT
- SUPPORTS.TXT

A2.2.2 Input files for inelastic dynamic analysis

The input files required additional to the files listed above for the nonlinear dynamic analysis are:

- INTEGRATION.TXT
- EARTHQUAKE.TXT
- DAMPING.TXT
- SPRING.TXT
- PRINT.TXT

A2.2.3 Output files for static and eigen value analyses

The output files created after the static and eigen value analyses are:

- INPUT.SEQ
- EIGENVALUE.SEQ
- S_DISPLACEMENTS.SEQ
- S_ELEMENT FORCES.SEQ

A2.2.4 Output files for inelastic dynamic analysis

The output files created additional to the files listed above for the nonlinear dynamic analysis are:

- D_TOTAL_SUPPORT_REACTIONS.SEQ
- D_DISPLACEMENTS.SEQ
- D_ELEMENT FORCES.SEQ
- D_HKOD.SEQ
- D_SPRING ROTATIONS.SEQ
- D_DISPLACEMENT_n.SEQ
- D_MOM_CHORDROT_n.SEQ
- D_FAILURE.SEQ
- D_YIELD.SEQ

A2.3. Input files

GEO.TXT: Structural geometry information

DATA:

Number of nodes, N

Number of elements, C

Number of coordinate commands, NCC

Node number, X coordinate, Y coordinate, Z coordinate –NCC times

Number of node generation commands, NNG

First node, Final node, Number of nodes to be created, Node number difference – NNG times

I end node number of first element, J end node number of first element – C times

Number of rigid end types, NRE

Rigid end length, at node I for first type	Rigid end length, at node J for first type	Number of elements which have this type of rigid ends, NEG	} NRE times
Element number - NEG times			

Y or N (y or n)

Value of Beta angle

Number of elements to which this beta angle will be assigned, BM

Element number – BM times

Y or N (y or n)

NOTES:

- Only one dimensional node generation is allowed
- Rigid end lengths have to be given as positive numbers at both ends
- Every beta angle information set starts with Y (or y)
- For structures with rectangular cross sections, beta is generally “0”, which is selected as default in the program. If you don’t want to assign a different beta angle than ‘0’ put N (or n) after ‘Rigid Ends’ commands.

DIAPHRAGM.TXT: Master-slave relation information

DATA:

Y or N (y or n)

Master node number

Direction in which the nodes will be slave to the master node

Number of slave nodes, NS

Node number – NS times

Y or N (y or n)

NOTES:

- If no diaphragm behavior will be assigned, put N (or n) at the first line of the file
- Put Y or (y) for each direction in which the nodes will be slave to the master node in order to give the necessary data

LOADS.TXT: Nodal and distributed loads

DATA:

Y or N (y or n)

Type of loading

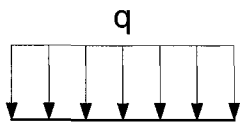
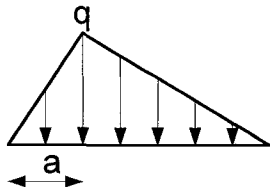
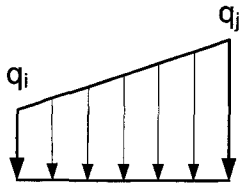
Table A2.1 Types of loading - SEQUAKE

Type	Explanation
1	Constant distributed loading in global X direction
2	Constant distributed loading in global Y direction
3	Constant distributed loading in global Z direction
4	Triangular distributed loading in global X direction
5	Triangular distributed loading in global Y direction
6	Triangular distributed loading in global Z direction
7	Trapezoidal distributed loading in global X direction
8	Trapezoidal distributed loading in global Y direction
9	Trapezoidal distributed loading in global Z direction

Number of elements to which this type of loading will be assigned, NDE

Loading Parameters

Table A2.2 Loading parameters - SEQUAKE

For Type	Loading Parameter
1	Value of distributed loading, q 
2	Value of distributed loading, q
3	Value of distributed loading, q
4	Max value of distributed loading, q Length of the left hand side portion of the triangle, a 
5	Max value of distributed loading, q Length of the left hand side portion of the triangle, a
6	Max value of distributed loading, q Length of the left hand side portion of the triangle, a
7	Value of distributed loading, q_i Value of distributed loading, q_j 
8	Value of distributed loading, q_i Value of distributed loading, q_j
9	Value of distributed loading, q_i Value of distributed loading, q_j

Element number – NDE times

Y or N (y or n)

Y or N (y or n)

Direction of the nodal loading

Number of nodes to which this type of loading will be assigned, NNE

Node number

Value of the nodal loading

Y or N (y or n)

NOTES:

- If there is no loading information put N (or n) at the first line of the file
- Give always first distributed loading information, then nodal loading information
- Every distributed loading information set starts with Y (or y)
- When all distributed loading information is assigned, put N (or n). If there is nodal loading information put Y (or y), else put again N (or n)
- Every nodal loading information set starts with Y (or y)
- Downward loading values in global Y direction should be entered as negative numbers.

MASS.TXT: Mass information

DATA:

Y or N (y or n)

Direction of the mass

Node number

Value of the mass

Y or N (y or n)

NOTES:

- Every mass assignment set starts with Y (or y)
- Nodes at which the masses are assigned can not be slaves

PROPERTY.TXT: Properties of the elements

DATA:

Number of property types, NPT

Modulus of elasticity, E

Poisson ratio, μ

Area of cross-section, A_x

Moment of inertia about x axis of the element, I_x

Moment of inertia about y axis of the element, I_y

Moment of inertia about z axis of the element, I_z

Reduced shear area, A_y

Reduced shear area, A_z

Number of elements to which this type of property will be assigned, NEP

Element numbers-NEP times

} NPT
times

NOTES:

- Moment of inertias should NOT be given as '0' even if the frame is planar.
- Reduced shear areas can be given as '0'

SUPPORTS.TXT: Support Information

DATA:

Number of support types, NST

Number of elements to which this type of support will be assigned, NES

Element numbers-NES times

Sup_X, Sup_Y, Sup_Z, Sup_RX, Sup_RY, Sup_RZ,

} NST
times

NOTES :

- Sup_ =1 when the support is fixed
- Sup_ =0 when the support is released

INTEGRATION.TXT

DATA:

Y or N (y or n)

Duration of the earthquake, T

Time interval, Δt

Number of steps, NOS

Ground motion record multiplication factor, FG

Newmark constant, γ

Newmark constant, β

NOTES :

- Y or N (y or n) put at the beginning of the file is in order to select if nonlinear dynamic analysis will be conducted or not. If selection is N (or n), program will not read the rest of the data given in this file and the following ones.
- Newmark time integration method is unconditionally stable when $\gamma=0.5$ and $\beta=0.25$.

EARTHQUAKE.TXT

DATA:

Direction of the earthquake

Ground motion record data

NOTES :

- Ground motion record data can NOT be given as time-acceleration pair.
- Ground motion record data can be given as accelerations or forces. This can be arranged with the 'Ground motion record multiplication factor, FG' given in INTEGRATION.TXT file.
- Ground motion record data have to be given as REAL numbers, NOT as integers.

DAMPING.TXT

DATA:

Mass-dependent damping ratio, α

Stiffness-dependent damping ratio, β

NOTES :

- Damping is ignored in the analysis put $\alpha=0.0$ and $\beta=0.0$

SPRING.TXT: Spring pair (one for each end of the element) information

DATA:

Number of spring pair information types, NST

(Repeat the following information sequence NST times)

Number of elements to which this type of spring information will be assigned, NSP

Element number - NSP times

Hysteretic model type, HST

Point of inflection factor, ξ

If Clough's model is assigned

Strain hardening ratio of end I under a positive moment about local y axis of the element

Strain hardening ratio of end I under a negative moment about local y axis of the element

Strain hardening ratio of end J under a positive moment about local y axis of the element

Strain hardening ratio of end J under a negative moment about local y axis of the element

Strain hardening ratio of end I under a positive moment about local z axis of the element

Strain hardening ratio of end I under a negative moment about local z axis of the element

Strain hardening ratio of end J under a positive moment about local z axis of the element

Strain hardening ratio of end J under a negative moment about local z axis of the element

Yield moment of end I under a positive moment about local y axis of the element

Yield moment of end I under a negative moment about local y axis of the element

Yield moment of end J under a positive moment about local y axis of the element
 Yield moment of end J under a negative moment about local y axis of the element
 Yield moment of end I under a positive moment about local z axis of the element
 Yield moment of end I under a negative moment about local z axis of the element
 Yield moment of end J under a positive moment about local z axis of the element
 Yield moment of end J under a negative moment about local z axis of the element

Initial spring stiffness at node I under a moment about local y axis of the element
 Initial spring stiffness at node J under a moment about local y axis of the element
 Initial spring stiffness at node I under a moment about local z axis of the element
 Initial spring stiffness at node J under a moment about local z axis of the element

If Saatcioglu-Sharbatdar's model is assigned

Slope2/Slope1 ratio of end I under a positive moment about local y axis of the element
 Slope3/Slope1 ratio of end I under a positive moment about local y axis of the element
 Slope2/Slope1 ratio of end I under a negative moment about local y axis of the element
 Slope3/Slope1 ratio of end I under a negative moment about local y axis of the element
 Slope2/Slope1 ratio of end J under a positive moment about local y axis of the element
 Slope3/Slope1 ratio of end J under a positive moment about local y axis of the element
 Slope2/Slope1 ratio of end J under a negative moment about local y axis of the element
 Slope3/Slope1 ratio of end J under a negative moment about local y axis of the element

Slope2/Slope1 ratio of end I under a positive moment about local z axis of the element
 Slope3/Slope1 ratio of end I under a positive moment about local z axis of the element
 Slope2/Slope1 ratio of end I under a negative moment about local z axis of the element
 Slope3/Slope1 ratio of end I under a negative moment about local z axis of the element
 Slope2/Slope1 ratio of end J under a positive moment about local z axis of the element
 Slope3/Slope1 ratio of end J under a positive moment about local z axis of the element
 Slope2/Slope1 ratio of end J under a negative moment about local z axis of the element
 Slope3/Slope1 ratio of end J under a negative moment about local z axis of the element

Moment1 of end I under a positive moment about local y axis of the element
Moment2 of end I under a positive moment about local y axis of the element
Moment3 of end I under a positive moment about local y axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end I under a negative moment about local y axis of the element
Moment2 of end I under a negative moment about local y axis of the element
Moment3 of end I under a negative moment about local y axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end J under a positive moment about local y axis of the element
Moment2 of end J under a positive moment about local y axis of the element
Moment3 of end J under a positive moment about local y axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end J under a negative moment about local y axis of the element
Moment2 of end J under a negative moment about local y axis of the element
Moment3 of end J under a negative moment about local y axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end I under a positive moment about local z axis of the element
Moment2 of end I under a positive moment about local z axis of the element
Moment3 of end I under a positive moment about local z axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end I under a negative moment about local z axis of the element
Moment2 of end I under a negative moment about local z axis of the element
Moment3 of end I under a negative moment about local z axis of the element
Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end J under a positive moment about local z axis of the element

Moment2 of end J under a positive moment about local z axis of the element

Moment3 of end J under a positive moment about local z axis of the element

Ratio of FRP rupture rotation to concrete crushing rotation

Moment1 of end J under a negative moment about local z axis of the element

Moment2 of end J under a negative moment about local z axis of the element

Moment3 of end J under a negative moment about local z axis of the element

Ratio of FRP rupture rotation to concrete crushing rotation

Initial spring stiffness at node I under a moment about local y axis of the element

Initial spring stiffness at node J under a moment about local y axis of the element

Initial spring stiffness at node I under a moment about local z axis of the element

Initial spring stiffness at node J under a moment about local z axis of the element

NOTES:

- Hysteretic model type HST=1 when Clough's model will be assigned
Hysteretic model type HST=2 when Saatcioglu-Sharbatdar's model will be assigned
- Point of inflection factor, ξ , defines the location of inflection point. When selected as '1.0', the end moments of the beam are assumed equal and the location of inflection point is assumed to be in the middle of the beam.
- If initial spring ration is given as '0.0', program will automatically assign a value equal to the effective stiffness of the element (to which the springs are assigned) times 10^8 .
- Slope 1, Slope 2 and Slope 3 are the slopes of three consecutive segments defined on the trilinear model of Saatcioglu-Sharbatdar.
- Moment1, Moment2 and Moment 3 are the moments which define the three segments on the trilinear model of Saatcioglu-Sharbatdar.

PRINT.TXT: Print information for selected nodes and elements

DATA:

Y or N (y or n)

Number of nodes of which the displacement responses will be printed, NNP

Node number, Direction – NNP times

Number of elements of which the moment-chord rotation responses will be printed, NMC

Element number, End, Direction –NMC times

NOTES:

- Put N (y or n) if there is no print assignment
- Chord rotations are calculated for frame elements assuming that the point of inflection is in the middle of the element.
- Directions in which the nodal displacement responses will be printed are the same as the global coordinate system directions.
- Directions in which the moment-chord rotation responses will be printed are:
 - 1: for rotation about y (local coordinate system)
 - 2: for rotation about z (local coordinate system)
- A maximum of 20 displacement responses and a maximum of 20 moment-chord rotation responses can be printed in one run.

A2.3.1 Example: Input files

A two-storey frame building is given as data entry example. The example is selected as a three-dimensional frame (even if the torsional effects are not included for the columns), in order to explain the 3D coordinate system. The geometry of the structure and element properties of the frame are shown in Fig. A2.1.

2D structures can be modeled by restraining their displacements in the perpendicular plane to the plane in which they are modeled. It is important to note that, any moment of inertia should be given as '0.0' even if there will not be any displacement in the restrained directions.

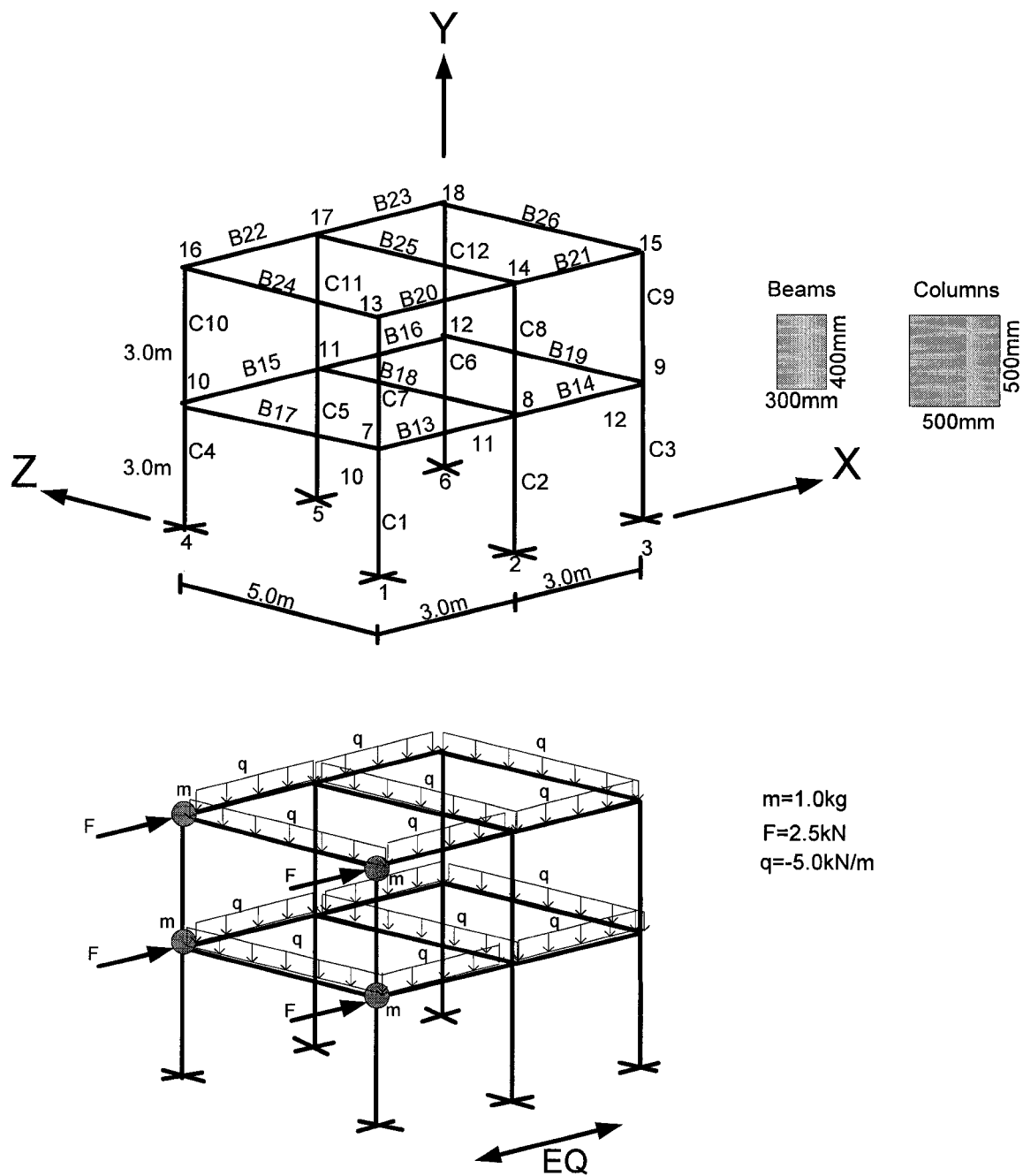


Figure A2.4 Data entry example: 2 storey frame building

GEO.TXT

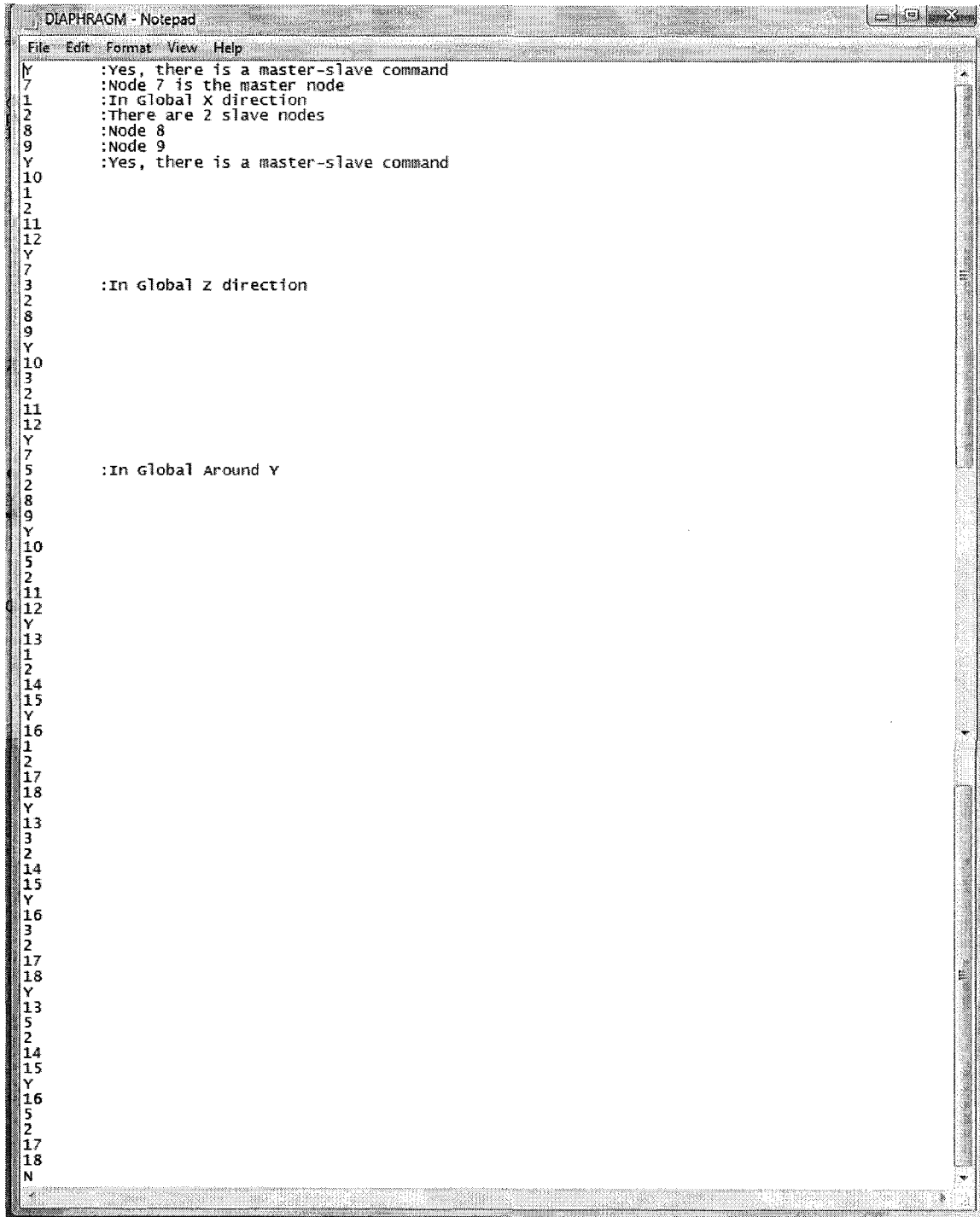
```

GEO - Notepad
File Edit Format View Help
18      :18 nodes
26      :26 elements
12      :18 coordinate commands
1,0.0,0.0,0.0 :Node1, X(1),Y(1), Z(1)
3,6.0,0.0,0.0
4,0.0,0.0,5.0
6,6.0,0.0,5.0
7,0.0,3.0,0.0
9,6.0,3.0,0.0
10,0.0,3.0,5.0
12,6.0,3.0,5.0
13,0.0,6.0,0.0
15,6.0,6.0,0.0
16,0.0,6.0,5.0
18,6.0,6.0,5.0 :Node18, X(18),Y(18), Z(18)
6        :There are 6 node generation commands
1,3,1,1  :First node:1,Final node:3,1 extra node between 1 and 3,increase node number by 1
4,6,1,1
7,9,1,1
10,12,1,1
13,15,1,1
16,18,1,1
1,7      :I end of element (1) is 1, J end of element (1) is 7
2,8
3,9
4,10
5,11
6,12
7,13
8,14
9,15
10,16
11,17
12,18
7,8
8,9
10,11
11,12
7,10
8,11
9,12
13,14
14,15
16,17
17,18
13,16
14,17
15,18    :I end of element (26) is 15, J end of node (26) is 18
2        :2 types of rigid end commands
0.25,0.25,14 :Type1:Rigidend length of I: 0.25,Rigidend length of J: 0.25, Type1 is assigned to
7        :Type 1 is assigned to Element 7
8
9
10
11
12
13
14
15
16
17
18
19
20      :Type 1 is assigned to Element 20
0.0,0.25,6 :Type2: Rigid end length of I: 0.0,Rigidend length of J : 0.25, Type2 is assigned
1        :Type 2 is assigned to Element 1
2
3
4
5
6        :Type 2 is assigned to Element 6
N        :There is no other beta angle than '0.0' for all elements

```

Figure A2.5 Data entry example: GEO.txt

DIAPHRAGM.TXT



```
DIAPHRAGM - Notepad
File Edit Format View Help
Y       :Yes, there is a master-slave command
7       :Node 7 is the master node
1       :In global x direction
2       :There are 2 slave nodes
8       :Node 8
9       :Node 9
Y       :Yes, there is a master-slave command
10
1
2
11
12
Y
7
3       :In global z direction
2
8
9
Y
10
3
2
11
12
Y
7
5       :In Global Around Y
2
8
9
Y
10
5
2
11
12
Y
13
1
2
14
15
Y
16
1
2
17
18
Y
13
3
2
14
15
Y
16
3
2
17
18
Y
13
5
2
14
15
Y
16
5
2
17
18
N
```

Figure A2.6 Data entry example: DIAPHRAGM.txt

LOADS.TXT

```
LOADS - Notepad
File Edit Format View Help
Y :Yes, there is distributed loading command
2 :In global Y direction
14 :The loading will be applied to 14 elements
-5.0 :q=-5.0 kN/m value of the distributed loading
13 :Element 13 is one of the 14 elements
14
15
16
17
18
19
20
21
22
23
24
25
26 :Element 26 is one of the 8 elements
N :There is no more distributed loading command
Y :Yes, there is nodal loading command
1 :In global X direction
4 :4 nodes and loading values will be listed
7 :Node 7
2.5 :F=2.5 kN
13 :Node 13
2.5 :F=2.5 kN
10 :Node 10
2.5 :F=2.5 kN
16 :Node 16
2.5 :F=2.5 kN
```

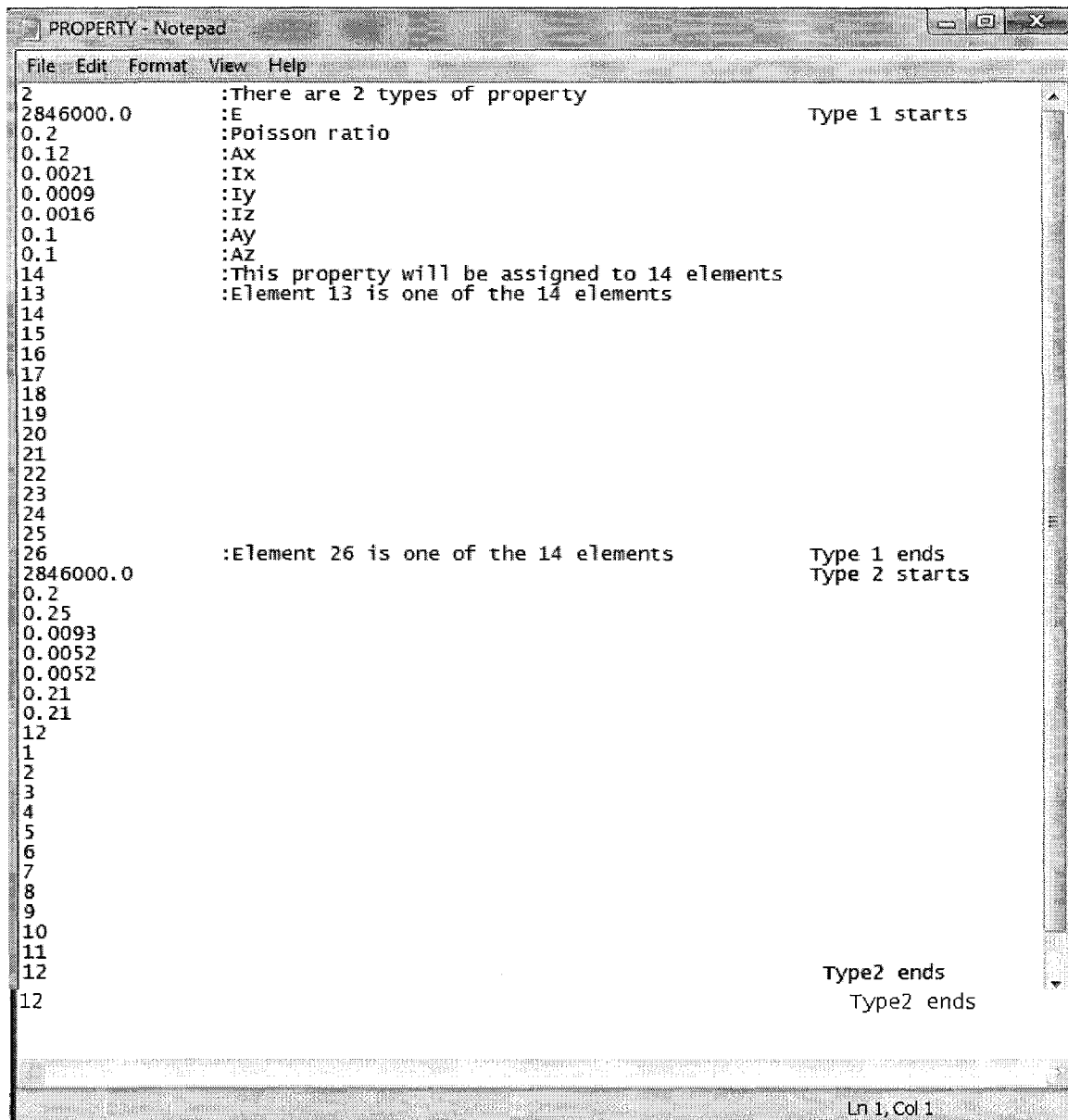
Figure A2.7 Data entry example: LOADSs.txt

MASS.TXT

```
MASS - Notepad
File Edit Format View Help
Y :yes, there is mass assignment command
1 :In global X direction
7 :Node 7
10.0 :m=10.0kg
Y
1
13
10.0
Y
1
10
10.0
Y
1
16
10.0
```

Figure A2.8 Data entry example: MASS.txt

PROPERTY.TXT



```
PROPERTY - Notepad
File Edit Format View Help
2      :There are 2 types of property
2846000.0 :E                                Type 1 starts
0.2      :Poisson ratio
0.12     :Ax
0.0021   :Ix
0.0009   :Iy
0.0016   :Iz
0.1      :Ay
0.1      :Az
14       :This property will be assigned to 14 elements
13       :Element 13 is one of the 14 elements
14
15
16
17
18
19
20
21
22
23
24
25
26      :Element 26 is one of the 14 elements      Type 1 ends
2846000.0 :E                                Type 2 starts
0.2      :Poisson ratio
0.25     :Ax
0.0093   :Ix
0.0052   :Iy
0.0052   :Iz
0.21     :Ay
0.21     :Az
12
1
2
3
4
5
6
7
8
9
10
11
12
12      :Element 12 is one of the 14 elements      Type2 ends
                                           Type2 ends

Ln 1, Col 1
```

Figure A2.9 Data entry example: PROPERTY.txt

SUPPORTS.TXT

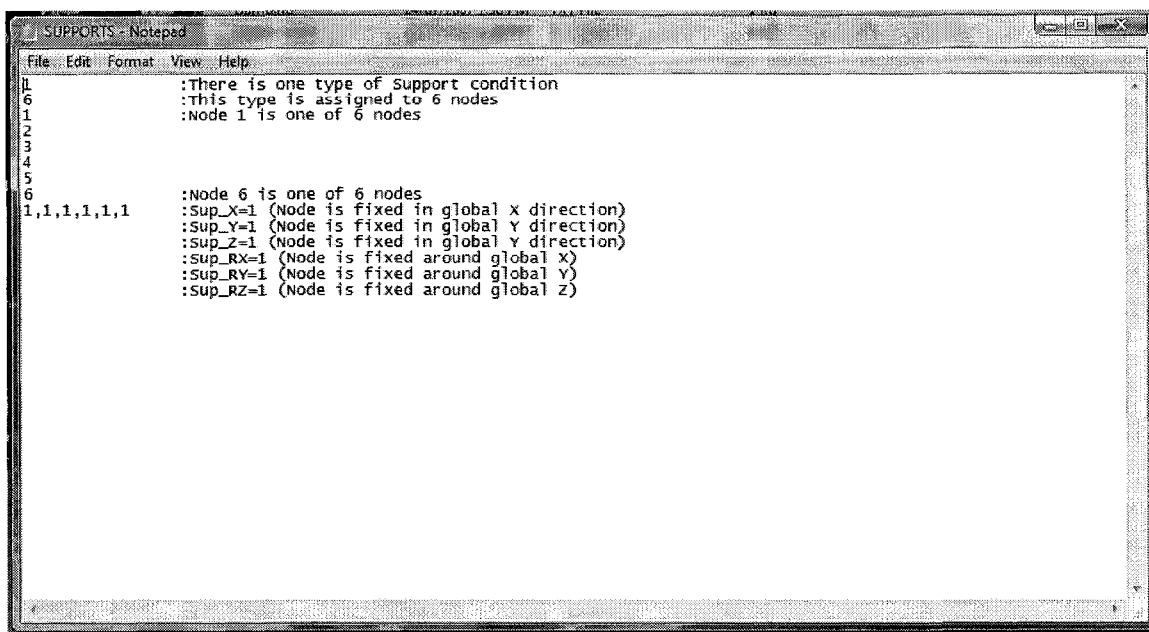


Figure A2.10 Data entry example: SUPPORTS.txt

INTEGRATION.TXT

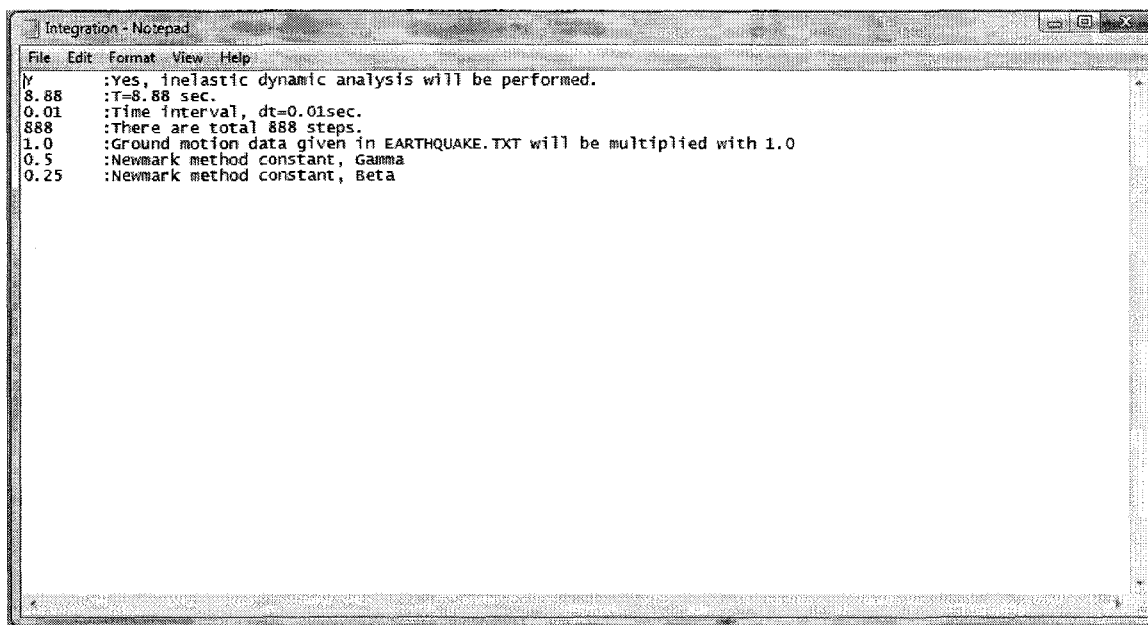


Figure A2.11 Data entry example: INTEGRATION.txt

EARTHQUAKE.TXT

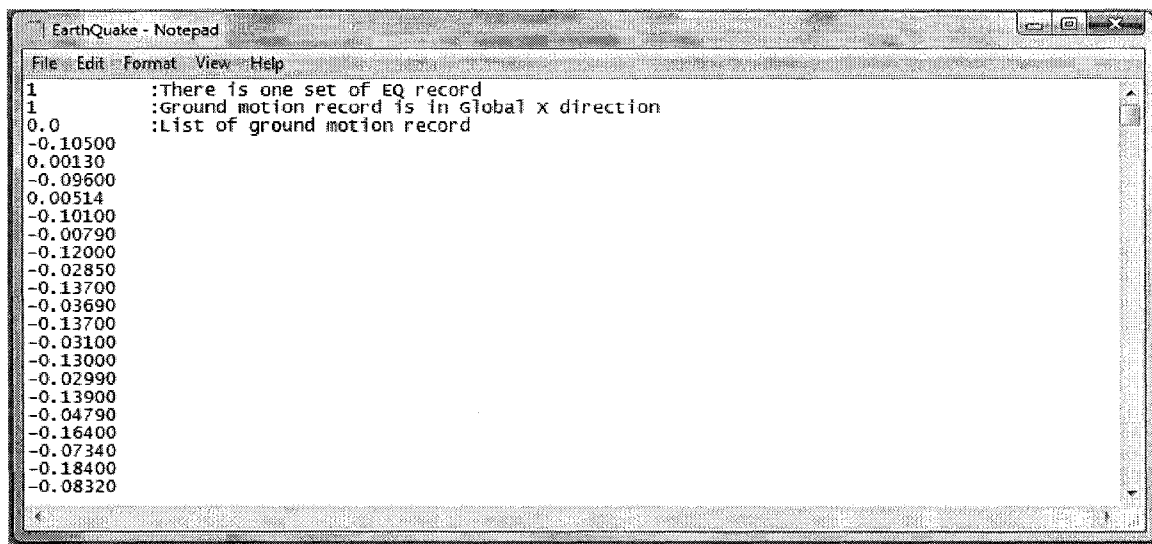


Figure A2.12 Data entry example: EARTHQUAKE.txt

DAMPING.TXT

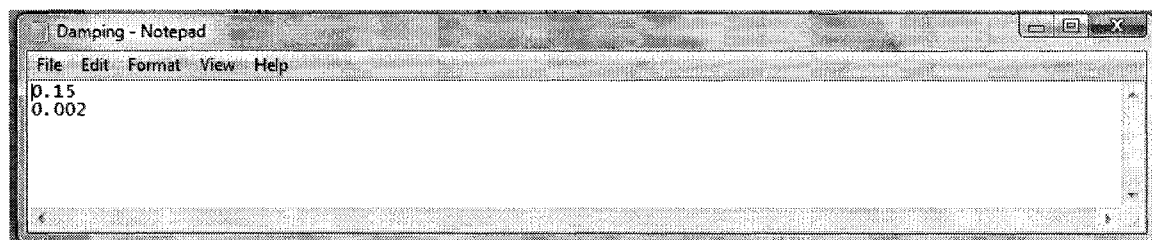


Figure A2.13 Data entry example: DAMPING.txt

SPRING.TXT: when Clough's model is assigned (steel reinforced concrete members)

```

SPRING - Notepad
File Edit Format View Help
2      :There are 2 types of spring information
1      :Type 1
12     :Type 1 is assigned to 12 elements
1      :Element 1 is one of 12 elements
2
3
4
5
6
7
8
9
10
11
12     :Element 12 is one of 10 elements
1      :Takeda's model is assigned
1.0    :Point of inflection ratio
.05    :Strain hardening ratio of end I under a positive moment about local y axis of the element
.05    :Strain hardening ratio of end I under a negative moment about local y axis of the element
.05    :Strain hardening ratio of end J under a positive moment about local y axis of the element
.05    :Strain hardening ratio of end J under a negative moment about local y axis of the element
.05    :Strain hardening ratio of end I under a positive moment about local z axis of the element
.05    :Strain hardening ratio of end I under a negative moment about local z axis of the element
.05    :Strain hardening ratio of end J under a positive moment about local z axis of the element
.05    :Strain hardening ratio of end J under a negative moment about local z axis of the element
5.0E3  :Yield moment of end I under a positive moment about local y axis of the element
-5.0E3 :Yield moment of end I under a negative moment about local y axis of the element
5.0E3  :Yield moment of end J under a positive moment about local y axis of the element
-5.0E3 :Yield moment of end J under a negative moment about local y axis of the element
5.0E3  :Yield moment of end I under a positive moment about local z axis of the element
-5.0E3 :Yield moment of end I under a negative moment about local z axis of the element
5.0E3  :Yield moment of end J under a positive moment about local z axis of the element
-5.0E3 :Yield moment of end J under a negative moment about local z axis of the element
0.0    :Initial spring stiffness at node I under a moment about local y axis of the element
0.0    :Initial spring stiffness at node J under a moment about local y axis of the element
0.0    :Initial spring stiffness at node I under a moment about local z axis of the element
0.0    :Initial spring stiffness at node J under a moment about local z axis of the element
2      :Type 2
14
13
14
15
16
17
18
19
20
21
22
23
24
25
26
1
1.0
.05
.05
.05
.05
.05
.05
.05
.05
.05
.05
2.0E3
-5.0E3
2.0E3
-5.0E3
2.0E3
-5.0E3
2.0E3
-5.0E3
0.0
0.0
0.0
0.0

```

Figure A2.14 Data entry example: SPRING.txt (Clough's model)

SPRING.TXT: when Saatcioglu-Sharbatdar's model is assigned (FRP reinforced
Concrete members)

```

SPRING - Notepad
File Edit Format View Help
2 :There are 2 types of spring information
1 :Type 1
4 :Type 1 is assigned to 4 elements
2 :Element 2 is one of 4 elements
4
9
10 :Element 10 is one of 4 elements
2 :Saatcioglu-sharbatdar's model is assigned
1.0 :Point of inflection ratio
.7 :Slope2/Slope1 ratio of end I under a positive moment about local y axis of the element
.5 :Slope3/Slope1 ratio of end I under a positive moment about local y axis of the element
.7 :Slope2/Slope1 ratio of end I under a negative moment about local y axis of the element
.5 :Slope3/Slope1 ratio of end I under a negative moment about local y axis of the element
.7 :Slope2/Slope1 ratio of end J under a positive moment about local y axis of the element
.5 :Slope3/Slope1 ratio of end J under a positive moment about local y axis of the element
.7 :Slope2/Slope1 ratio of end J under a negative moment about local y axis of the element
.5 :Slope3/Slope1 ratio of end J under a negative moment about local y axis of the element
.7 :Slope2/Slope1 ratio of end I under a positive moment about local z axis of the element
.5 :Slope3/Slope1 ratio of end I under a negative moment about local z axis of the element
.7 :Slope2/Slope1 ratio of end I under a negative moment about local z axis of the element
.5 :Slope3/Slope1 ratio of end I under a positive moment about local z axis of the element
.7 :Slope2/Slope1 ratio of end J under a positive moment about local z axis of the element
.5 :Slope3/Slope1 ratio of end J under a positive moment about local z axis of the element
.7 :Slope2/Slope1 ratio of end J under a negative moment about local z axis of the element
.5 :Slope3/Slope1 ratio of end J under a negative moment about local z axis of the element
2.0E2 :Moment1 of end I under a positive moment about local y axis of the element
3.0E2 :Moment2 of end I under a positive moment about local y axis of the element
4.0E2 :Moment3 of end I under a positive moment about local y axis of the element
1.2 :FRP rupture rotation/concrete crushing rotation - end I - positive moment about y axis
-5.0E2 :Moment1 of end I under a negative moment about local y axis of the element
-6.0E2 :Moment2 of end I under a negative moment about local y axis of the element
-7.0E2 :Moment3 of end I under a negative moment about local y axis of the element
1.15 :FRP rupture rotation/concrete crushing rotation - end I - negative moment about y axis
2.0E2 :Moment1 of end J under a positive moment about local y axis of the element
3.0E2 :Moment2 of end J under a positive moment about local y axis of the element
4.0E2 :Moment3 of end J under a positive moment about local y axis of the element
1.2 :FRP rupture rotation/concrete crushing rotation - end J - positive moment about y axis
-5.0E2 :Moment1 of end J under a negative moment about local y axis of the element
-6.0E2 :Moment2 of end J under a negative moment about local y axis of the element
-7.0E2 :Moment3 of end J under a negative moment about local y axis of the element
1.15 :FRP rupture rotation/concrete crushing rotation - end J - negative moment about y axis
2.0E2 :Moment1 of end I under a positive moment about local z axis of the element
3.0E2 :Moment2 of end I under a positive moment about local z axis of the element
4.0E2 :Moment3 of end I under a positive moment about local z axis of the element
1.2 :FRP rupture rotation/concrete crushing rotation - end I - positive moment about z axis
-5.0E2 :Moment1 of end I under a negative moment about local z axis of the element
-6.0E2 :Moment2 of end I under a negative moment about local z axis of the element
-7.0E2 :Moment3 of end I under a negative moment about local z axis of the element
1.15 :FRP rupture rotation/concrete crushing rotation - end I - negative moment about z axis
2.0E2 :Moment1 of end J under a positive moment about local z axis of the element
3.0E2 :Moment2 of end J under a positive moment about local z axis of the element
4.0E2 :Moment3 of end J under a positive moment about local z axis of the element
1.2 :FRP rupture rotation/concrete crushing rotation - end J - positive moment about z axis
-5.0E2 :Moment1 of end J under a negative moment about local z axis of the element
-6.0E2 :Moment2 of end J under a negative moment about local z axis of the element
-7.0E2 :Moment3 of end J under a negative moment about local z axis of the element
1.15 :FRP rupture rotation/concrete crushing rotation - end J - negative moment about z axis
0.0 :Initial spring stiffness at node I under a moment about local y axis of the element
0.0 :Initial spring stiffness at node J under a moment about local y axis of the element
0.0 :Initial spring stiffness at node I under a moment about local z axis of the element
0.0 :Initial spring stiffness at node J under a moment about local z axis of the element
2 :Type 2

```

Figure A2.15 Data entry example: SPRING.txt (Saatcioglu-Sharbatdar's model)

6
1
3
5
6
7
8
2
1.0
.7
.5
.7
.5
.7
.5
.7
.5
.7
.5
.7
.5
.7
.5
2.0E2
3.0E2
4.0E2
1.2
-5.0E2
-6.0E2
-7.0E2
1.15
2.0E2
3.0E2
4.0E2
1.2
-5.0E2
-6.0E2
-7.0E2
1.15
2.0E2
3.0E2
4.0E2
1.2
-5.0E2
-6.0E2
-7.0E2
1.15
2.0E2
3.0E2
4.0E2
1.2
-5.0E2
-6.0E2
-7.0E2
1.15
2.0E2
3.0E2
4.0E2
1.2
-5.0E2
-6.0E2
-7.0E2
1.15
0.0
0.0
0.0
0.0

Figure A2.15 cont'd Data entry example: SPRING.txt (Saatcioglu-Sharbatdar's model)

PRINT.TXT

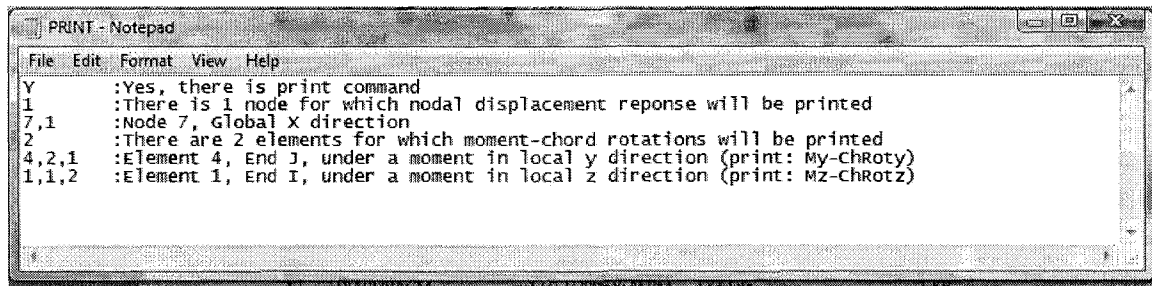


Figure A2.16 Data entry example: PRINT.txt

A2.4. Output files

INPUT.SEQ lists the input data given by the user

EIGENVALUE.SEQ lists the eigen values (periods) for each mode

S_DISPLACEMENTS.SEQ lists nodal displacements of each node (static analysis)

S_ELEMENT FORCES.SEQ lists internal forces of each element node (static analysis)

D_TOTAL_SUPPORT_REACTIONS.SEQ lists total support reaction forces at the end of each time step. In a conventional structure as in the example, Reactions in X and Z directions would give the base shear forces.

D_DISPLACEMENTS.SEQ lists nodal displacements of each node at the end of each time step

D_ELEMENT FORCES.SEQ lists internal forces of each element at the end of each time step

D_HKOD.SEQ lists the sequence of codes which show the status on the hysteretic models at the end of each time step. The codes for hysteretic models are illustrated in Fig. A2.5 and A2.6.

D_SPRING ROTATIONS.SEQ lists spring rotations at the end of each element at the end of each time step

D_DISPLACEMENT_n.SEQ lists displacements of the node selected in PRINT.TXT, at the end of each time step.

D_MOM_CHORDROT_n.SEQ lists moment-chord rotation pair as well as the status codes for the element selected in PRINT.TXT. The list includes the values calculated when there are changes during time steps; therefore it is generally longer than the other output files which have 'number of step' times output data.

D_YIELD.SEQ lists the element number and the time step, if any yielding occurs for steel reinforced concrete elements

D_FAILURE.SEQ lists the element number and the time step, if any failure occurs for FRP reinforced concrete elements. Failures can be either ‘concrete crushing’ or ‘FRP rupturing’. A warning also printed on the SEQUAKE.EXE screen, if any failure occurs.

D_DUCTILITY_RATIOS lists the ductility ratios of each element under positive and negative moment. In the program, ductility ratio is defined as the ratio of maximum chord rotation to chord rotation at yield.

$$\mu_{\theta} = \frac{\theta_{max}}{\theta_y} \quad (A2.1)$$

This file is only for steel reinforces elements.

Status codes for Clough's model

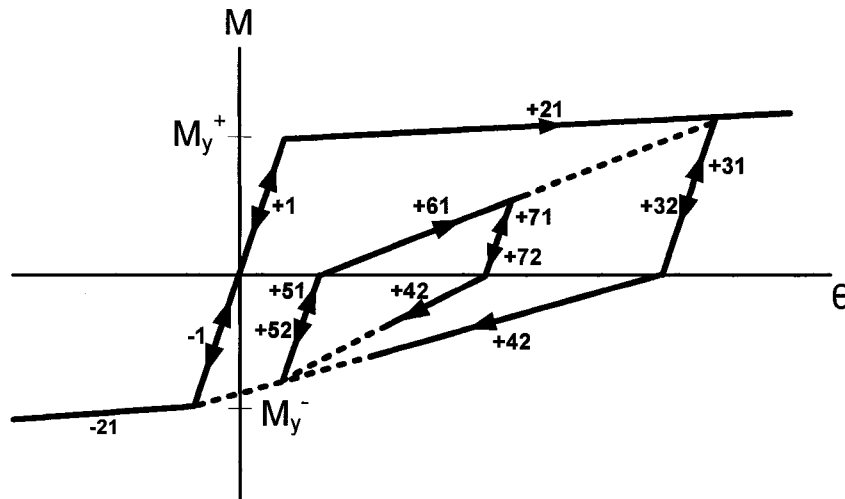


Figure A2.17 Status codes of moment-spring rotation relationship (Clough's Model)

NOTES:

- The positive status codes on the under positive moments become negative under negative moments. The shape and rules are same.
- When there is a stiffness change due to unloading and reloading, the status codes start with another number other than shown in the figure. For example, +21

becomes first +320 for a very small interval (much smaller than the time step defined in the input) then +32. The same sequence will be observed for:

```
+31 / +310 / +21
+42 / +510 / +51
+52 / +520 / +42
+61 / +610 / +21
+61 / +720 / +72
```

```
-31 / -310 / -21
-21 / -320 / -32
-42 / -510 / -51
-52 / -520 / -42
-61 / -610 / -21
-61 / -720 / -72
```

- User should be aware of that these extra codes (310, 320, 510, 520, 610, 720) occur within a time step and they are only printed in D_MOM_CHORDROT_n.SEQ files. In D_HKOD.SEQ the status codes at the end of the time steps are only printed and they do not include those.

Status codes for Saatcioglu-Sharbatdar's model

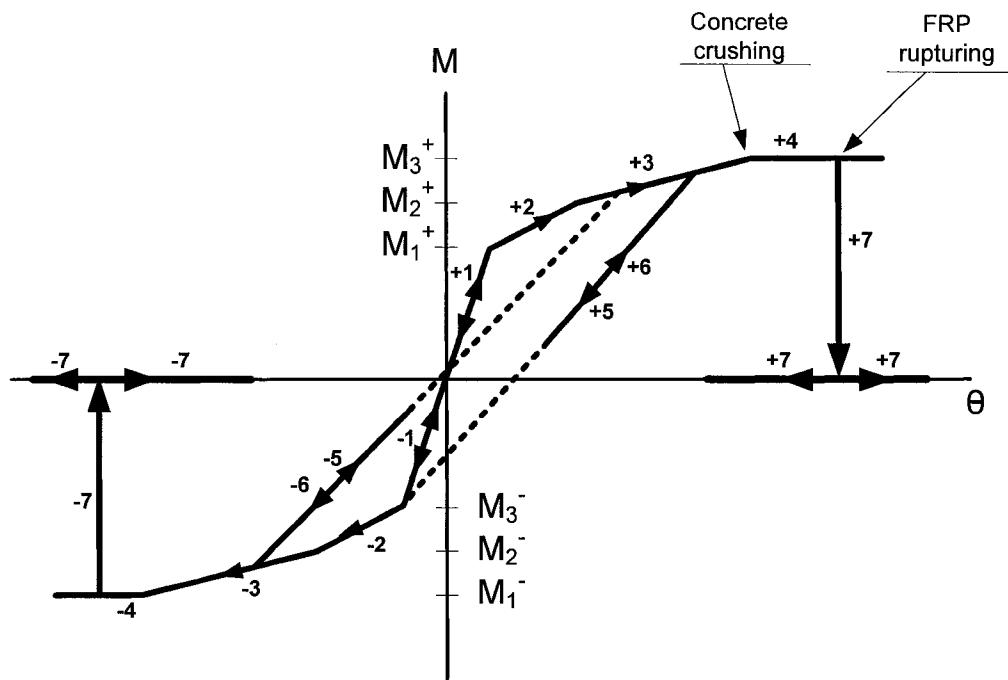


Figure A2.18 Status codes of moment-spring rotation relationship
(Saatcioglu - Sharbatdar model)

NOTES:

- As in the Clough's model, when there is a stiffness change due to unloading and reloading, the status codes start with another number other than shown in the figure. Consequently, +2 becomes first +50 for a very small interval (much smaller than the time step defined in the input) then +5. The same sequence will be observed for:

+3 / +50 / +5

+6 / +60 / +2

+6 / +60 / +3

-2 / -50 / -5

-3 / -50 / -5

-6 / -60 / -2

-6 / -60 / -3

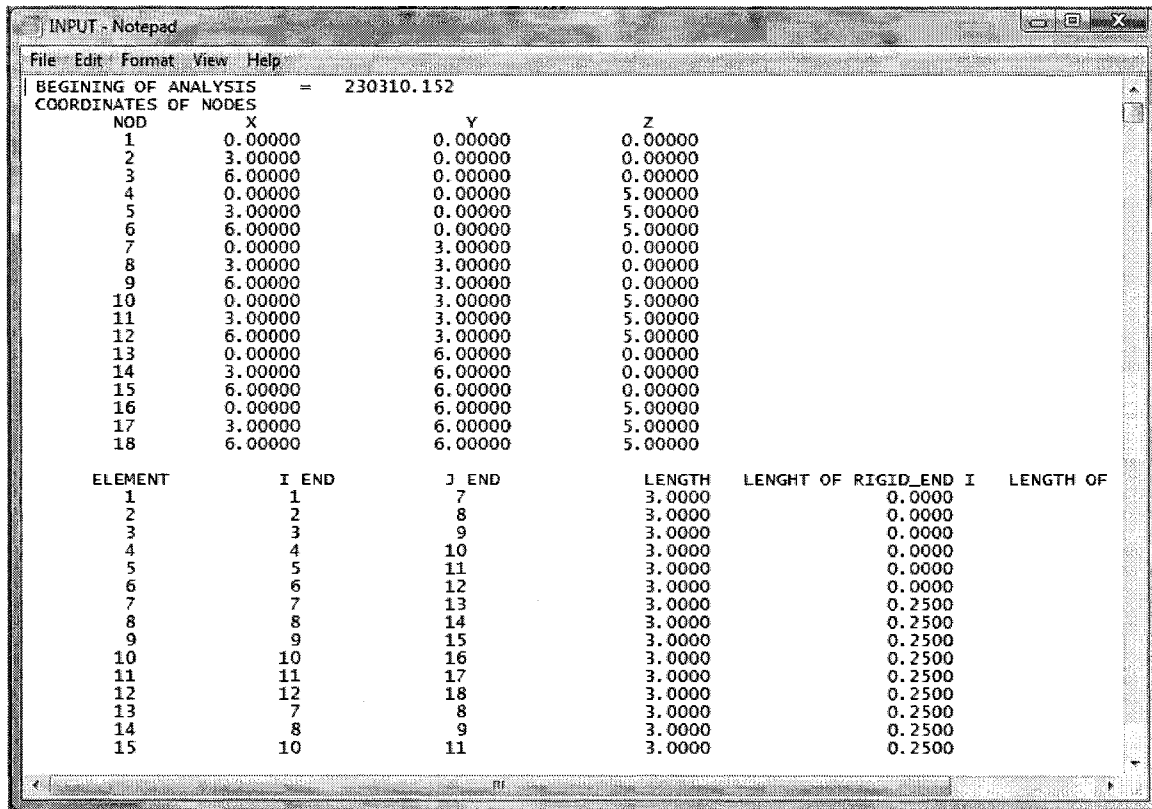
- User should be aware of that these extra codes (50, 60) occur within a time step and they are only printed in D_MOM_CHORDROT_n.SEQ files. In D_HKOD.SEQ the status codes at the end of the time steps are printed only and they do not include those.
- Failure Modes:
 - Concrete Crushing: Status Code is '+4' or '-4'. This moment is represented with 'Moment 3' which is defined by user in the SPRING.TXT input file. After concrete crushing the primary curve continues as a flat line (with zero stiffness) until FRP rupturing.
 - FRP rupturing: Status Code is '+7' or '-7'. The location of FRP rupturing is represented with its rotation value instead of a moment value, as it will be equal to 'Moment 3'. The ratio of the FRP rupturing rotation to concrete crushing rotation is defined by user in the SPRING.TXT input file. FRP rupturing is followed by a sudden perpendicular drop from the final moment value to zero. In following steps, the model follows the zero rotation axis depending on the direction of displacement.

One should be aware of that sudden perpendicular drop in moment when element fails can not be observed in the moment-chord rotation relationship, since it is not possible to control the contribution of the elastic beam to the total stiffness of the element. The moment-chord rotation relationship would show an angled drop toward zero point. However, one can find the point where failure occurs by following the status codes.

A2.4.1 Example: Output files

Output files of the example designed with steel reinforced concrete are listed below. D_HKOD.SEQ and D_FAILURE.SEQ are also added for the same example when FRP reinforced concrete is used.

INPUT.SEQ

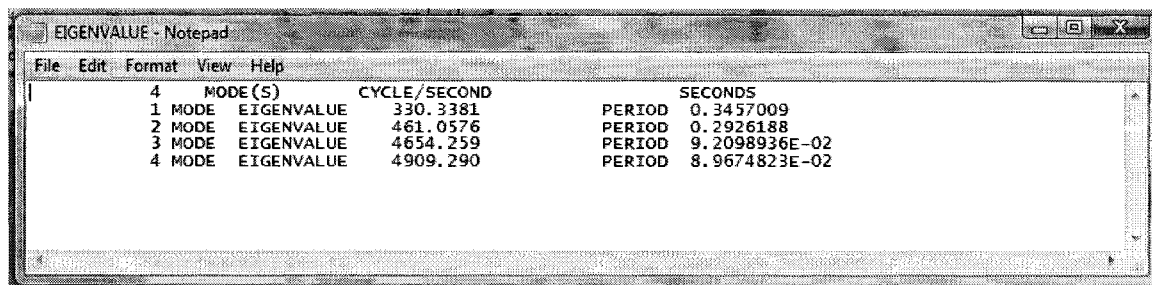


```
INPUT - Notepad
File Edit Format View Help
BEGINNING OF ANALYSIS = 230310.152
COORDINATES OF NODES
NOD      X      Y      Z
1      0.00000  0.00000  0.00000
2      3.00000  0.00000  0.00000
3      6.00000  0.00000  0.00000
4      0.00000  0.00000  5.00000
5      3.00000  0.00000  5.00000
6      6.00000  0.00000  5.00000
7      0.00000  3.00000  0.00000
8      3.00000  3.00000  0.00000
9      6.00000  3.00000  0.00000
10     0.00000  3.00000  5.00000
11     3.00000  3.00000  5.00000
12     6.00000  3.00000  5.00000
13     0.00000  6.00000  0.00000
14     3.00000  6.00000  0.00000
15     6.00000  6.00000  0.00000
16     0.00000  6.00000  5.00000
17     3.00000  6.00000  5.00000
18     6.00000  6.00000  5.00000

ELEMENT  I END  J END  LENGTH  LENGHT OF RIGID_END I  LENGTH OF
1        1     7     3.0000  0.0000
2        2     8     3.0000  0.0000
3        3     9     3.0000  0.0000
4        4    10     3.0000  0.0000
5        5    11     3.0000  0.0000
6        6    12     3.0000  0.0000
7        7    13     3.0000  0.2500
8        8    14     3.0000  0.2500
9        9    15     3.0000  0.2500
10       10   16     3.0000  0.2500
11       11   17     3.0000  0.2500
12       12   18     3.0000  0.2500
13       7     8     3.0000  0.2500
14       8     9     3.0000  0.2500
15       10    11     3.0000  0.2500
```

Figure A2.19 Output example: INPUT.seq

EIGENVALUE.SEQ



```
EIGENVALUE - Notepad
File Edit Format View Help
4 MODE(S) CYCLE/SECOND SECONDS
1 MODE EIGENVALUE 330.3381 PERIOD 0.3457009
2 MODE EIGENVALUE 461.0576 PERIOD 0.2926188
3 MODE EIGENVALUE 4654.259 PERIOD 9.2098936E-02
4 MODE EIGENVALUE 4909.290 PERIOD 8.9674823E-02
```

Figure A2.20 Output example: EIGENVALUE.seq

S_DISPLACEMENTS.SEQ

NODAL DISPLACEMENTS		X	Y	Z	rX	rY	rZ
1	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
2	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
3	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
4	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
5	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
6	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
7	0.4579E-03	-0.1452E-03	-0.3022E-04	0.1100E-03	-0.2203E-19	-0.2041E-03	-0.1312E-03
8	0.4579E-03	-0.2144E-03	-0.3022E-04	0.1100E-03	-0.2203E-19	-0.1159E-03	-0.2041E-03
9	0.4579E-03	-0.1622E-03	-0.3022E-04	0.1100E-03	-0.2203E-19	-0.1159E-03	-0.2041E-03
10	0.4579E-03	-0.1452E-03	0.3022E-04	-0.1100E-03	-0.2250E-19	-0.1312E-03	-0.1159E-03
11	0.4579E-03	-0.2144E-03	0.3022E-04	-0.1100E-03	-0.2250E-19	-0.1159E-03	-0.2041E-03
12	0.4579E-03	-0.1622E-03	0.3022E-04	-0.1100E-03	-0.2250E-19	-0.1159E-03	-0.2041E-03
13	0.8996E-03	-0.2118E-03	0.4096E-04	0.3438E-03	0.3606E-19	-0.2020E-04	-0.7570E-04
14	0.8996E-03	-0.3126E-03	0.4096E-04	0.3438E-03	0.3606E-19	0.8843E-05	-0.2020E-04
15	0.8996E-03	-0.2345E-03	0.4096E-04	0.3438E-03	0.3606E-19	-0.2020E-04	-0.7570E-04
16	0.8996E-03	-0.2118E-03	-0.4096E-04	-0.3438E-03	0.3516E-19	0.8843E-05	-0.2020E-04
17	0.8996E-03	-0.3126E-03	-0.4096E-04	-0.3438E-03	0.3516E-19	-0.2020E-04	-0.7570E-04
18	0.8996E-03	-0.2345E-03	-0.4096E-04	-0.3438E-03	0.3516E-19	0.8843E-05	-0.2020E-04

Figure A2.21 Output example: S_DISPLACEMENTS.seq

S_ELEMENT FORCES.SEQ

ELEMENT END INTERNAL FORCES		FX	FY	FZ	MX	MY	MZ
1	1	37.5592	0.9856	1.6304	0.0000	-1.6500	2.4534
	7	-37.5592	-0.9856	-1.6304	0.0000	-2.8337	0.2569
2	2	55.4824	1.9100	1.6304	0.0000	-1.6500	3.3322
	8	-55.4824	-1.9100	-1.6304	0.0000	-2.8337	1.9203
3	3	41.9583	2.1044	1.6304	0.0000	-1.6500	3.5170
	9	-41.9583	-2.1044	-1.6304	0.0000	-2.8337	2.2701
4	4	37.5592	0.9856	-1.6304	0.0000	1.6500	2.4534
	10	-37.5592	-0.9856	1.6304	0.0000	2.8337	0.2569
5	5	55.4824	1.9100	-1.6304	0.0000	1.6500	3.3322
	11	-55.4824	-1.9100	1.6304	0.0000	2.8337	1.9203
6	6	41.9583	2.1044	-1.6304	0.0000	1.6500	3.5170
	12	-41.9583	-2.1044	1.6304	0.0000	2.8337	2.2701
7	7	18.9716	-1.7084	6.2176	0.0000	-6.3880	-2.1478
	13	-18.9716	1.7084	-6.2176	0.0000	-9.1561	-2.1231

Figure A2.22 Output example: S_ELEMENT FORCES.seq

STEP	REACTION X	REACTION Y	REACTION Z
1	-9.9227	270.0000	0.0000
2	-9.7158	270.0000	0.0000
3	-9.4430	270.0000	0.0000
4	-9.1717	270.0000	0.0000
5	-8.9371	270.0000	0.0000
6	-8.7307	270.0000	0.0000
7	-8.5099	270.0000	0.0000
8	-8.2228	270.0000	0.0000
9	-7.8357	270.0000	0.0000
10	-7.3533	270.0000	0.0000
11	-6.8194	270.0000	0.0000
12	-6.2983	270.0000	0.0000
13	-5.8487	270.0000	0.0000
14	-5.4982	270.0000	0.0000
15	-5.2326	270.0000	0.0000
16	-5.0063	270.0000	0.0000
17	-4.7657	270.0000	0.0000
18	-4.4766	270.0000	0.0000
19	-4.1428	270.0000	0.0000
20	-3.8070	270.0000	0.0000
21	-3.5331	270.0000	0.0000

Figure A2.23 Output example: D_TOTAL_SUPPORT_REACTIONS.seq

D_DISPLACEMENTS.SEQ

DYNAMIC ANALYSIS-ELEMENT FORCES							
ELEMENT	NODE	FX	FY	FZ	MX	MY	MZ
1	1	37.5592	0.9733	1.6304	0.0000	-1.6500	2.4139
	7	-37.5592	-0.9733	-1.6304	0.0000	-2.8337	0.2290
2	2	55.4824	1.8959	1.6304	0.0000	-1.6500	3.2892
	8	-55.4824	-1.8959	-1.6304	0.0000	-2.8337	1.8857
3	3	41.9583	2.0922	1.6304	0.0000	-1.6500	3.4776
	9	-41.9583	-2.0922	-1.6304	0.0000	-2.8337	2.2423
4	4	37.5592	0.9733	-1.6304	0.0000	1.6500	2.4139
	10	-37.5592	-0.9733	1.6304	0.0000	2.8337	0.2290
5	5	55.4824	1.8959	-1.6304	0.0000	1.6500	3.2892
	11	-55.4824	-1.8959	1.6304	0.0000	2.8337	1.8857
6	6	41.9583	2.0922	-1.6304	0.0000	1.6500	3.4776
	12	-41.9583	-2.0922	1.6304	0.0000	2.8337	2.2423
7	7	18.9716	-1.7046	6.2176	0.0000	-6.3880	-2.1301
	13	-18.9716	1.7046	-6.2176	0.0000	-9.1561	-2.1218
8	8	27.9472	1.3433	6.2176	0.0000	-6.3880	1.3599
	14	-27.9472	-1.3433	-6.2176	0.0000	-9.1561	2.0066
9	9	20.5812	2.8722	6.2176	0.0000	-6.3880	2.8650
	15	-20.5812	-2.8722	-6.2176	0.0000	-9.1561	4.3250
10	10	18.9716	-1.7046	-6.2176	0.0000	6.3880	-2.1301
	16	-18.9716	1.7046	6.2176	0.0000	9.1561	-2.1218

Figure A2.24 Output example: D_DISPLACEMENTS.seq

D_ELEMENT FORCES.SEQ

ELEMENT		DYNAMIC ANALYSIS-ELEMENT FORCES			MOMENTS		
		FX	FY	FZ	MX	MY	MZ
1	1	37.5592	0.9733	1.6304	0.0000	-1.6500	2.4139
	7	-37.5592	-0.9733	-1.6304	0.0000	-2.8337	0.2290
2	2	55.4824	1.8959	1.6304	0.0000	-1.6500	3.2892
	8	-55.4824	-1.8959	-1.6304	0.0000	-2.8337	1.8857
3	3	41.9583	2.0922	1.6304	0.0000	-1.6500	3.4776
	9	-41.9583	-2.0922	-1.6304	0.0000	-2.8337	2.2423
4	4	37.5592	0.9733	-1.6304	0.0000	1.6500	2.4139
	10	-37.5592	-0.9733	1.6304	0.0000	2.8337	0.2290
5	5	55.4824	1.8959	-1.6304	0.0000	1.6500	3.2892
	11	-55.4824	-1.8959	1.6304	0.0000	2.8337	1.8857
6	6	41.9583	2.0922	-1.6304	0.0000	1.6500	3.4776
	12	-41.9583	-2.0922	1.6304	0.0000	2.8337	2.2423
7	7	18.9716	-1.7046	6.2176	0.0000	-6.3880	-2.1301
	13	-18.9716	1.7046	-6.2176	0.0000	-9.1561	-2.1218
8	8	27.9472	1.3433	6.2176	0.0000	-6.3880	1.3599
	14	-27.9472	-1.3433	-6.2176	0.0000	-9.1561	2.0066
9	9	20.5812	2.8722	6.2176	0.0000	-6.3880	2.8650
	15	-20.5812	-2.8722	-6.2176	0.0000	-9.1561	4.3250
10	10	18.9716	-1.7046	-6.2176	0.0000	6.3880	-2.1301
	16	-18.9716	1.7046	6.2176	0.0000	9.1561	-2.1218

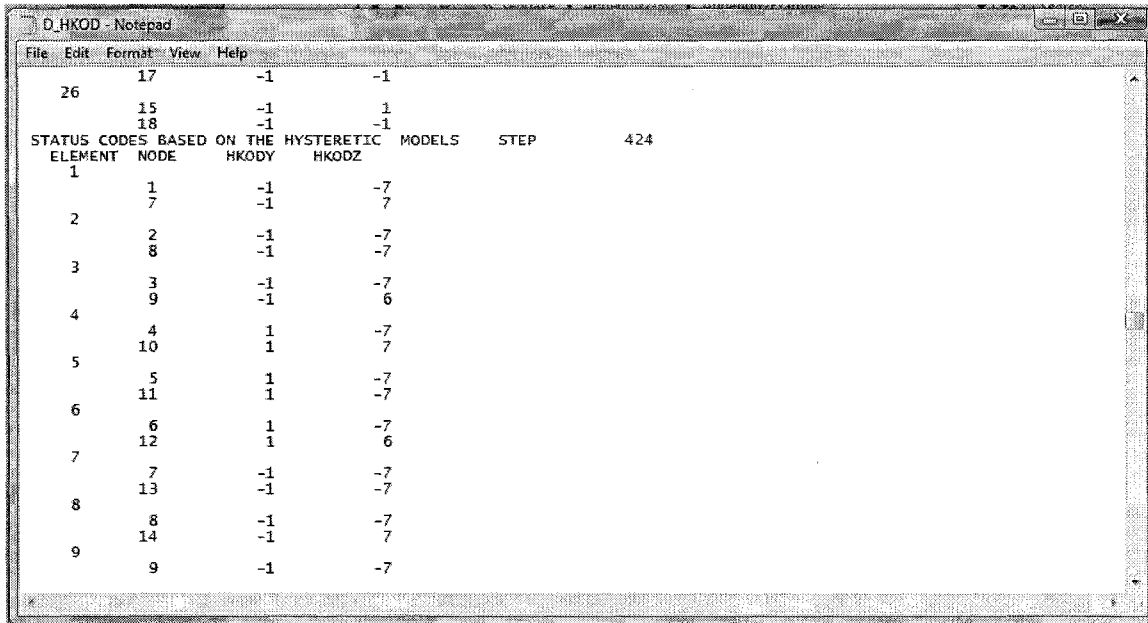
Figure A2.25 Output example: D_ELEMENT FORCES.seq

D_HKOD.SEQ *for steel reinforced concrete members*

ELEMENT		STATUS CODES BASED ON THE Hysteretic Models		STEP
		HKODY	HKODZ	
1	1	-1	-72	328
	7	-1	-1	
2	2	-1	-72	
	8	-1	-1	
3	3	-1	-1	
	9	-1	-1	
4	4	1	-72	
	10	1	-1	
5	5	1	-72	
	11	1	-1	
6	6	1	-1	
	12	1	-1	
7	7	-1	-1	
	13	-1	-1	
8	8	-1	-1	
	14	-1	-1	
9	9	-1	-1	
	15	-1	-1	

Figure A2.26 Output example: D_HKOD.seq (steel reinforced members)

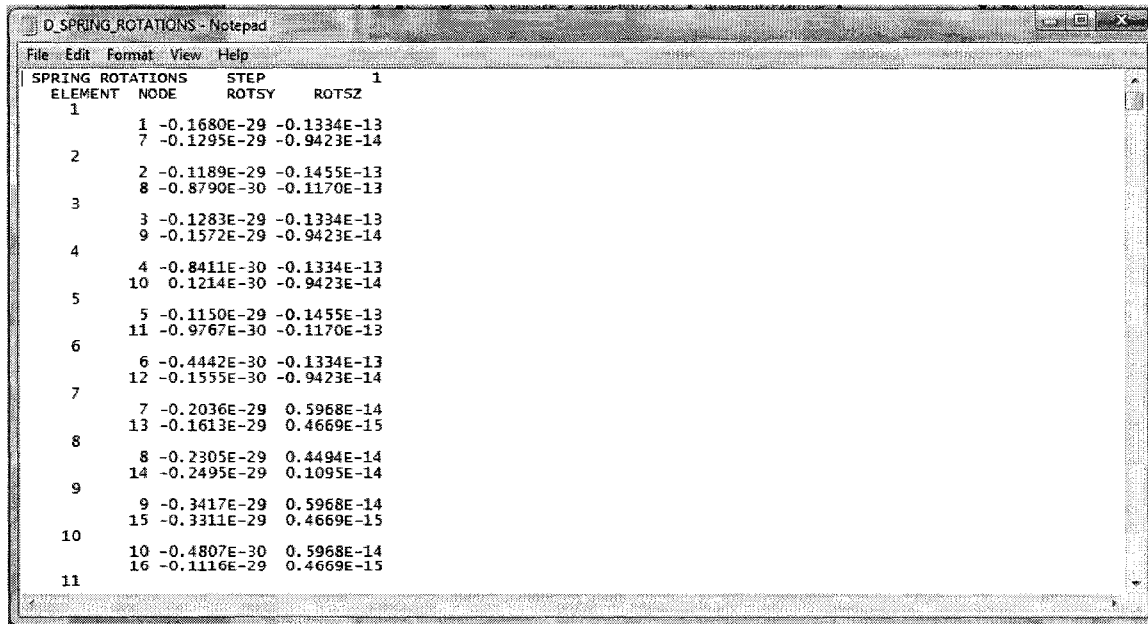
D_HKOD.SEQ *for FRP reinforced concrete members*



STATUS	CODES BASED ON THE	HYSTERETIC	MODELS	STEP	424
ELEMENT	NODE	HKODY	HKODZ		
1	1	-1	-7		
2	7	-1	7		
3	2	-1	-7		
4	8	-1	-7		
5	3	-1	-7		
6	9	-1	6		
7	4	1	-7		
8	10	1	7		
9	5	1	-7		
	11	1	-7		
	6	1	-7		
	12	1	6		
	7	-1	-7		
	13	-1	-7		
	8	-1	-7		
	14	-1	7		
	9	-1	-7		

Figure A2.27 Output example: D_HKOD.seq (FRP reinforced members)

D_SPRING ROTATIONS.SEQ



SPRING ROTATIONS	STEP	1
ELEMENT	NODE	ROTSY ROTSZ
1	1	-0.1680E-29 -0.1334E-13
	7	-0.1295E-29 -0.9423E-14
2	2	-0.1189E-29 -0.1455E-13
	8	-0.8790E-30 -0.1170E-13
3	3	-0.1283E-29 -0.1334E-13
	9	-0.1572E-29 -0.9423E-14
4	4	-0.8411E-30 -0.1334E-13
	10	0.1214E-30 -0.9423E-14
5	5	-0.1150E-29 -0.1455E-13
	11	-0.9767E-30 -0.1170E-13
6	6	-0.4442E-30 -0.1334E-13
	12	-0.1555E-30 -0.9423E-14
7	7	-0.2036E-29 0.5968E-14
	13	-0.1613E-29 0.4669E-15
8	8	-0.2305E-29 0.4494E-14
	14	-0.2495E-29 0.1095E-14
9	9	-0.3417E-29 0.5968E-14
	15	-0.3311E-29 0.4669E-15
10	10	-0.4807E-30 0.5968E-14
	16	-0.1116E-29 0.4669E-15
11		

Figure A2.28 Output example: D_SPRING ROTATIONS.seq

D_DISPLACEMENT_n.SEQ

DISPLACEMENT IN X DIRECTION
0 -4.579371258458982E-004
1 -4.603827360716616E-004
2 -4.671609015934696E-004
3 -4.767787226783977E-004
4 -4.875996793306363E-004
5 -4.987284754926720E-004
6 -5.102731080684093E-004
7 -5.231186349447032E-004
8 -5.383746845010867E-004
9 -5.567174797982027E-004
10 -5.779028262721051E-004
11 -6.007189222234294E-004
12 -6.233952078672520E-004
13 -6.442277510756206E-004
14 -6.622033528114850E-004
15 -6.773305678140886E-004
16 -6.904612041049711E-004
17 -7.027729009490204E-004
18 -7.151447786895943E-004
19 -7.276650280818766E-004
20 -7.395181434982286E-004
21 -7.493151515535984E-004
22 -7.556927852955212E-004
23 -7.579399204994401E-004
24 -7.563841557889305E-004
25 -7.523494463999737E-004
26 -7.477345463934827E-004
27 -7.443415933015766E-004
28 -7.432300274807191E-004
29 -7.444269740497144E-004
30 -7.470715142055067E-004

Figure A2.29 Output example: D_DISPLACEMENT_n.seq

D_MOM_CHORDROT_n.SEQ

D_MOM_CHORDROT_1 - Notepad

File Edit Format View Help

ELEMENT	4	END	2	AROUND	Z	H-Code
	chord	Rz			Mz	
7.	735600188265091E-006			0.228961388891254		1
6.	503588112745385E-006			0.192495803404503		1
4.	967720878016202E-006			0.147036591553279		1
3.	604920433242843E-006			0.106699879675434		1
2.	658262985969618E-006			7.868033444845285E-002		1
2.	054148765678232E-006			6.079952046819656E-002		1
1.	473688808857409E-006			4.361883482584483E-002		1
5.	286407202155278E-007			1.564692403842518E-002		1
-1.	036311023099124E-006			-3.067314271449789E-002		1
-3.	205115944330733E-006			-9.486629701121643E-002		-1
-5.	688887980526942E-006			-0.168381973777053		-1
-8.	058830201607264E-006			-0.238528470210547		-1
-9.	938418361614366E-006			-0.294161271293045		-1
-1.	117660593200064E-005			-0.330809641543399		-1
-1.	191740890770117E-005			-0.352736223900841		-1
-1.	251825680308359E-005			-0.370520359891846		-1
-1.	337199951528322E-005			-0.395789777679227		-1
-1.	471384662602178E-005			-0.435506304407381		-1
-1.	649403927732157E-005			-0.488197157523795		-1
-1.	838200990788134E-005			-0.544078066317737		-1
-1.	990544791650314E-005			-0.589169392970302		-1
-2.	065812617072896E-005			-0.611447464564618		-1
-2.	048872730291950E-005			-0.606433529215925		-1
-1.	958694297661501E-005			-0.579742156546062		-1
-1.	842181678938476E-005			-0.545256286295664		-1
-1.	756234523671988E-005			-0.519817304000047		-1
-1.	744547289648835E-005			-0.516358069794123		-1
-1.	818942818783600E-005			-0.538377955649150		-1
-1.	954631120628091E-005			-0.578539521179058		-1
-2.	100852268090911E-005			-0.621818640424111		-1
-2.	203135567915679E-005			-0.652092860033961		-1

Figure A2.30 Output example: D_MOM_CHORDROT_n.seq

D_YIELD.SEQ *for steel reinforced concrete members*

```

D_YIELD - Notepad
File Edit Format View Help
STEP      176      YIELDING AT ELEMENT 13      NODE 1 UNDER MZ
STEP      176      YIELDING AT ELEMENT 15      NODE 1 UNDER MZ
STEP      222      YIELDING AT ELEMENT 14      NODE 2 UNDER MZ
STEP      222      YIELDING AT ELEMENT 16      NODE 2 UNDER MZ
STEP      274      YIELDING AT ELEMENT 14      NODE 1 UNDER MZ
STEP      274      YIELDING AT ELEMENT 16      NODE 1 UNDER MZ
STEP      289      YIELDING AT ELEMENT 13      NODE 2 UNDER MZ
STEP      289      YIELDING AT ELEMENT 15      NODE 2 UNDER MZ
STEP      290      YIELDING AT ELEMENT 2       NODE 1 UNDER MZ
STEP      290      YIELDING AT ELEMENT 5       NODE 1 UNDER MZ
STEP      291      YIELDING AT ELEMENT 1       NODE 1 UNDER MZ
STEP      291      YIELDING AT ELEMENT 4       NODE 1 UNDER MZ
STEP      293      YIELDING AT ELEMENT 21      NODE 2 UNDER MZ
STEP      293      YIELDING AT ELEMENT 23      NODE 2 UNDER MZ
STEP      294      YIELDING AT ELEMENT 20      NODE 2 UNDER MZ
STEP      294      YIELDING AT ELEMENT 22      NODE 2 UNDER MZ

```

Figure A2.31 Output example: D_YIELD.seq (steel reinforced members)

D_FAILURE.SEQ *for FRP reinforced concrete members*

STEP	143	CONCRETE CRUSHING AT ELEMENT	8	NODE 2 UNDER MZ
STEP	143	CONCRETE CRUSHING AT ELEMENT	11	NODE 2 UNDER MZ
STEP	143	FRP RUPTURING AT ELEMENT	8	NODE 2 UNDER MZ
STEP	143	CONCRETE CRUSHING AT ELEMENT	2	NODE 1 UNDER MZ
STEP	143	CONCRETE CRUSHING AT ELEMENT	5	NODE 1 UNDER MZ
STEP	143	FRP RUPTURING AT ELEMENT	11	NODE 2 UNDER MZ
STEP	155	FRP RUPTURING AT ELEMENT	5	NODE 1 UNDER MZ
STEP	155	FRP RUPTURING AT ELEMENT	2	NODE 1 UNDER MZ
STEP	155	CONCRETE CRUSHING AT ELEMENT	1	NODE 1 UNDER MZ
STEP	155	CONCRETE CRUSHING AT ELEMENT	4	NODE 1 UNDER MZ
STEP	155	CONCRETE CRUSHING AT ELEMENT	3	NODE 1 UNDER MZ
STEP	155	CONCRETE CRUSHING AT ELEMENT	6	NODE 1 UNDER MZ
STEP	155	FRP RUPTURING AT ELEMENT	1	NODE 1 UNDER MZ
STEP	155	FRP RUPTURING AT ELEMENT	4	NODE 1 UNDER MZ
STEP	155	FRP RUPTURING AT ELEMENT	6	NODE 1 UNDER MZ
STEP	156	FRP RUPTURING AT ELEMENT	3	NODE 1 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	14	NODE 2 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	16	NODE 2 UNDER MZ
STEP	158	FRP RUPTURING AT ELEMENT	14	NODE 2 UNDER MZ
STEP	158	FRP RUPTURING AT ELEMENT	16	NODE 2 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	13	NODE 2 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	15	NODE 2 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	5	NODE 2 UNDER MZ
STEP	158	CONCRETE CRUSHING AT ELEMENT	2	NODE 2 UNDER MZ
STEP	159	FRP RUPTURING AT ELEMENT	5	NODE 2 UNDER MZ
STEP	159	FRP RUPTURING AT ELEMENT	13	NODE 2 UNDER MZ
STEP	159	FRP RUPTURING AT ELEMENT	15	NODE 2 UNDER MZ
STEP	159	FRP RUPTURING AT ELEMENT	2	NODE 2 UNDER MZ
STEP	160	CONCRETE CRUSHING AT ELEMENT	8	NODE 1 UNDER MZ
STEP	160	CONCRETE CRUSHING AT ELEMENT	11	NODE 1 UNDER MZ
STEP	160	FRP RUPTURING AT ELEMENT	8	NODE 1 UNDER MZ
STEP	160	FRP RUPTURING AT ELEMENT	11	NODE 1 UNDER MZ

Figure A2.32 Output example: D_FAILURE.seq (FRP reinforced members)

D_DUCTILITY_RATIOS.SEQ *for steel reinforced concrete members*

ELEMENT	NODE	DUCTILITY RATIO Y	DUCTILITY RATIO Z
1	1 POSITIVE	0.0000	0.0508
	1 NEGATIVE	0.0003	1.0569
	7 POSITIVE	0.0000	0.0007
	7 NEGATIVE	0.0006	0.0214
2	2 POSITIVE	0.0000	0.0595
	2 NEGATIVE	0.0003	1.1832
	8 POSITIVE	0.0000	0.0051
	8 NEGATIVE	0.0006	0.0028
3	3 POSITIVE	0.0000	0.0040
	3 NEGATIVE	0.0003	0.0467
	9 POSITIVE	0.0000	0.0092
	9 NEGATIVE	0.0006	0.0020
4	4 POSITIVE	0.0003	0.0508
	4 NEGATIVE	0.0000	1.0569
	10 POSITIVE	0.0006	0.0007
	10 NEGATIVE	0.0000	0.0214
5	5 POSITIVE	0.0003	0.0595
	5 NEGATIVE	0.0000	1.1832
	11 POSITIVE	0.0006	0.0051
	11 NEGATIVE	0.0000	0.0028
6	6 POSITIVE	0.0003	0.0040
	6 NEGATIVE	0.0000	0.0467
	12 POSITIVE	0.0006	0.0092
	12 NEGATIVE	0.0000	0.0020

Figure A2.33 Output example: D_ DUCTILITY_RATIOS.seq (steel reinforced members)

APPENDIX 3

SEQUAKE User Interface

A user interface has been developed in order to simplify data entry procedure of SEQUAKE software. One can use this interface for all input information described in Appendix 2, as well as for displaying the structure geometry, static loads and analysis results.

Installation of the interface (Fig.A3.1) is consisted of three steps; i) save **SEQUAKE** user interface (UI) to any directory on your computer, ii) create a new folder called “Sequake” at the same directory where **SEQUAKE** (UI) is stored , iii) save SeQuake analysis software (**SeQuakef**) and graphical interface tool (**mfcgcl**) to folder “Sequake”. Run the software by double clicking on **SEQUAKE** icon.

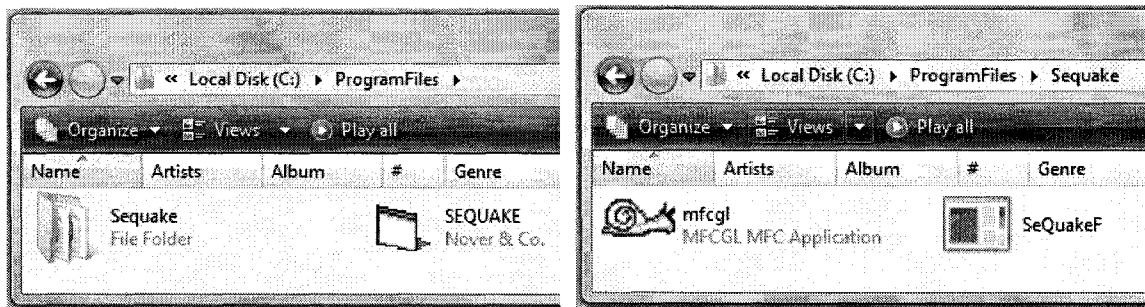


Figure A3.1 Installation files for SEQUAKE

User interface has been developed using Visual Basic 6.0 software and it contains a series of windows, where the input data can be entered. Data entry can be saved with any file name (filename) and on any directory chosen by the user, and reopened for further modifications.

During the analyses, the set of input and output files described in Appendix 2 are saved under the folder “Sequake” automatically. Additionally, the analysis results are stored

under a folder named “filename.SEQ(Results)” at the same directory where the input data is saved (Fig.A3.2). Here “filename” is the input data name that user has assigned.



Figure A3.2 Input and Output files for SEQUAKE

The results are displayed through the main menu of the user interface; they can also be accessed directly from the results folder (filename.SEQ(Results)). After analysis of an input file, the results are replaced with new ones. In order to save space it is recommended to delete the results folder whenever the associated data file is deleted.

If preferred the procedure described in Appendix 2 can still be used by using SeQuakeF.exe directly. The software will run after double clicking on SeQuakeF.exe; the input files described in Appendix 2 need to be stored at the same directory as SeQuakeF.exe

A3.1. Data entry example: 2 storey frame building

Two storey frame building (Fig.A3.3) used in Appendix 2 is selected to explain the functions of the user interface.

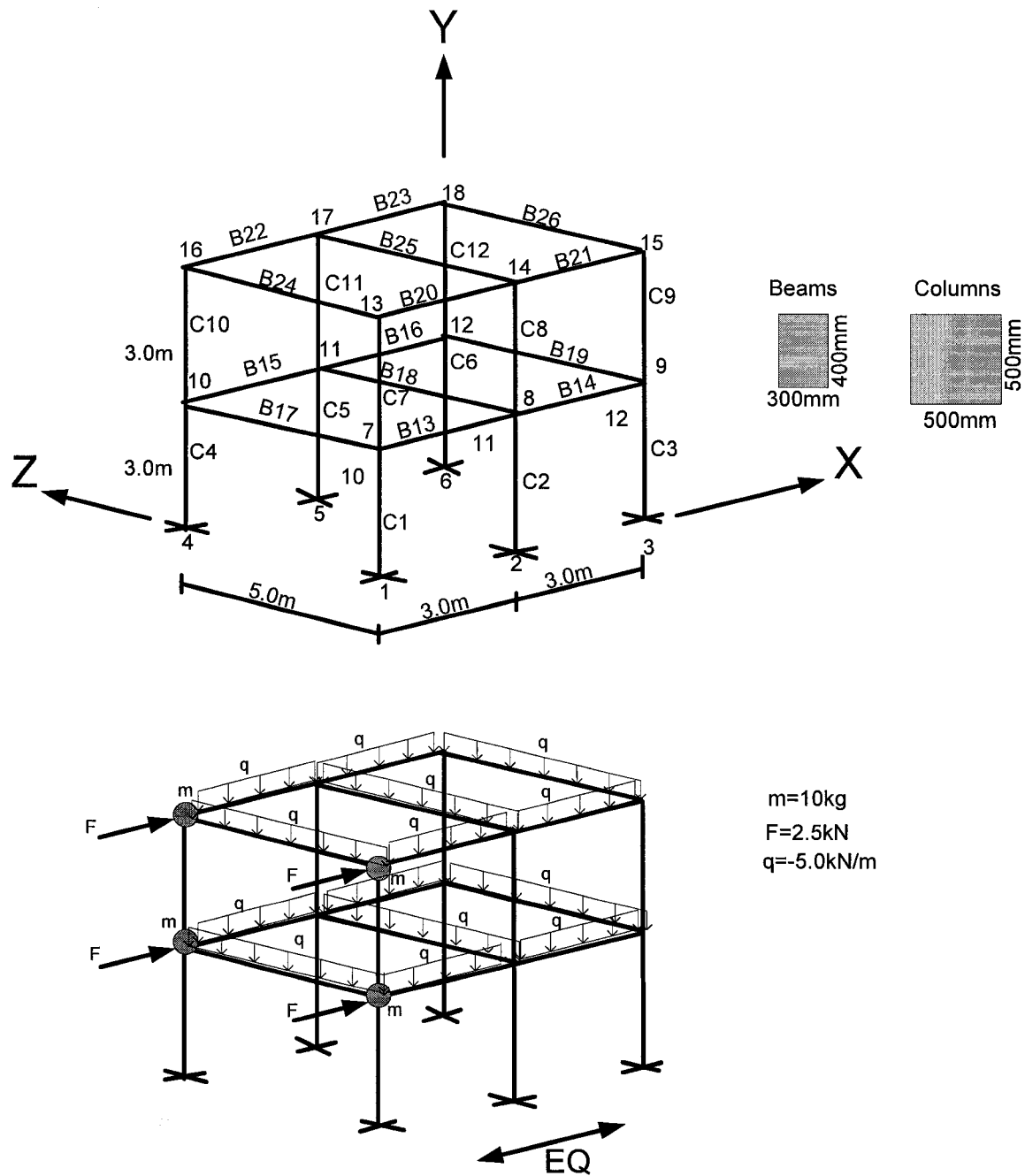


Figure A3.3 Data entry example: 2 storey frame building

The software opens with the Main Window shown in Fig. A3.4.

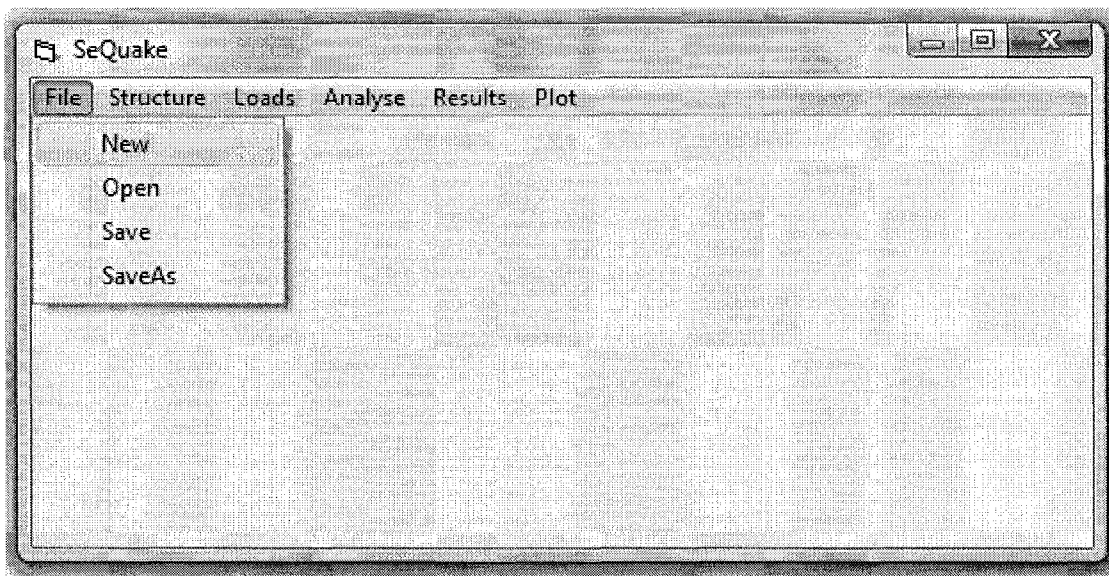


Figure A3.4 Main Window

Geometry information (Fig.A3.5) can be entered by clicking on **Geometry** under the Structure Menu. The tables are generated based on the node and element numbers and the information can be entered by clicking on each cell. Deleting is achieved by double clicking on a cell. In order to give rigid end and beta angle information appropriate checkboxes needed to be selected. Accordingly the tables having the same number of rows as the element number will be displayed.

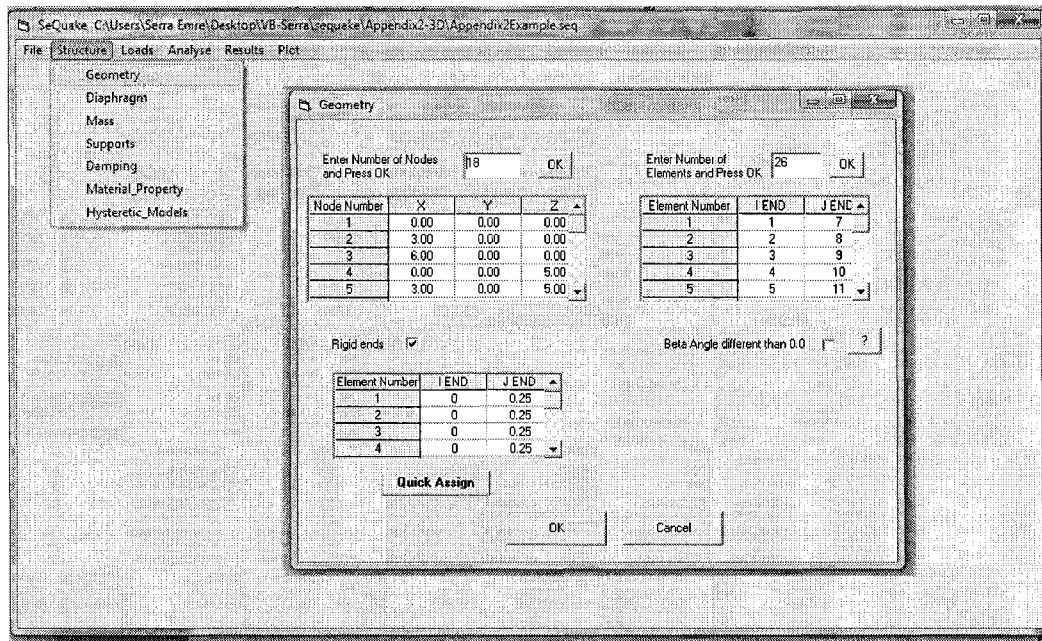


Figure A3.5 Geometry Information

Quick Assign button is developed in several windows to simplify the data input for nodes and elements as shown in Fig. A3.6. Thus, the nodes or elements with consecutive numbers which have the same property (rigid ends, support conditions, masses etc.) can be chosen directly from the formed linked to the **Quick Assign** button.

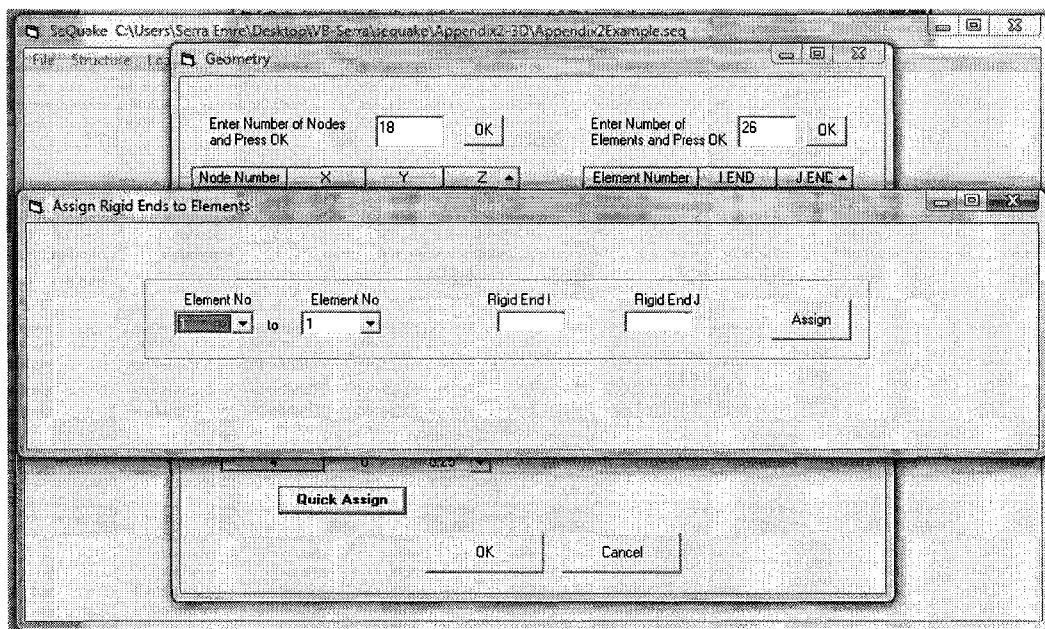


Figure A3.6 Quick Assign for Rigid End Information

Master/Slave node information can be entered from the window shown in Fig. A3.7. The first column shows the node numbers of the structure and the next six columns show the master nodes in each direction. If a node has no master node, then it is the master of itself in each direction.

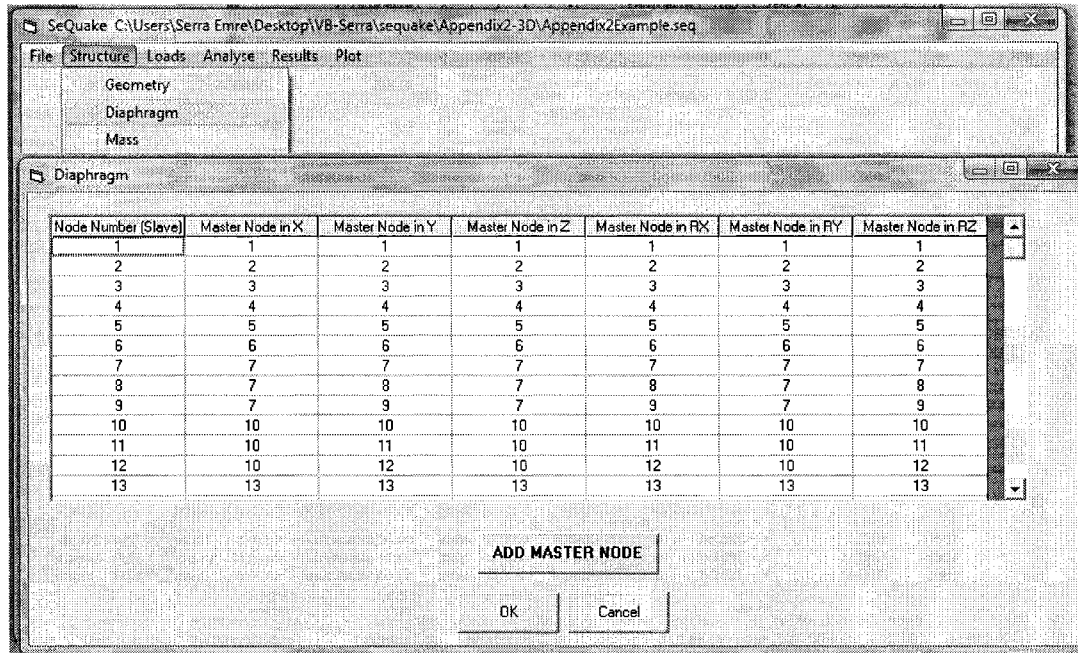


Figure A3.7 Master/Slave Node Information

Changes can be done by either clicking on the table or using the window shown in Fig. A3.8 which can be displayed by clicking on **ADD MASTER NODE**.

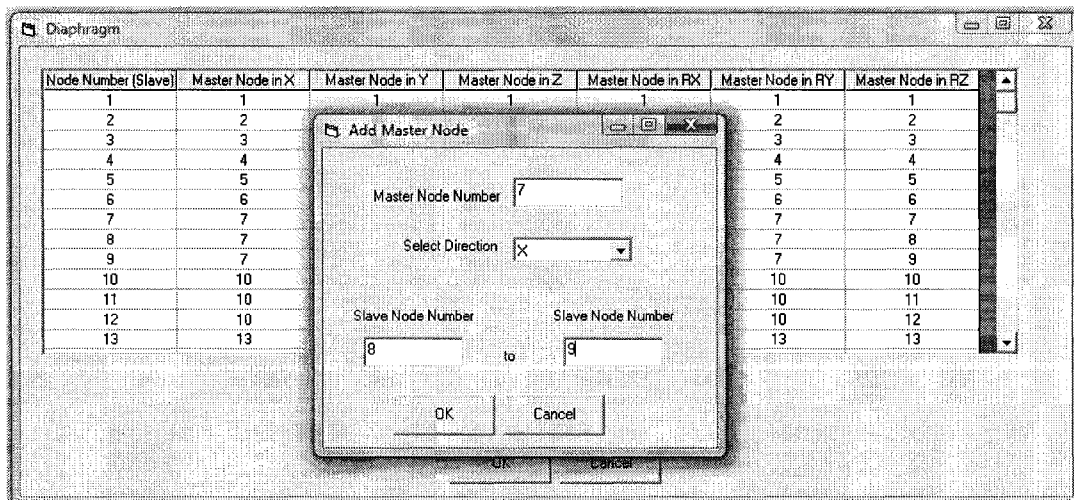
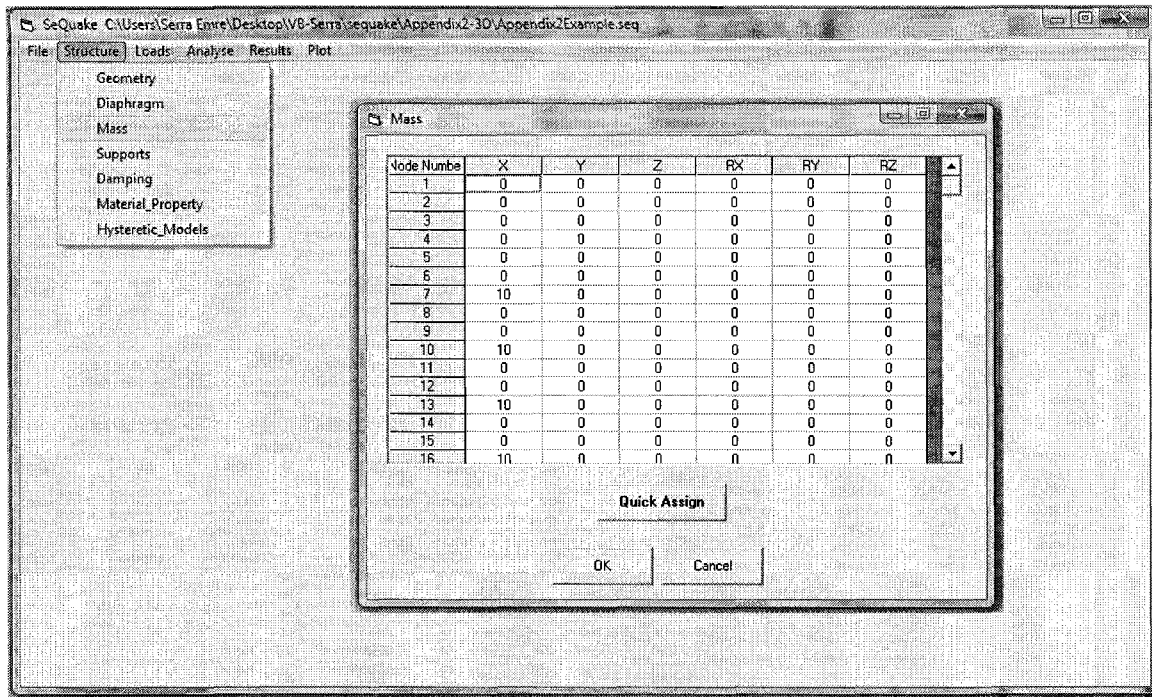


Figure A3.8 Add Master Node Information

Following similar steps, mass and support information can be entered as shown in Figs. A3.9 and A3.10.



A3.9 Mass Information

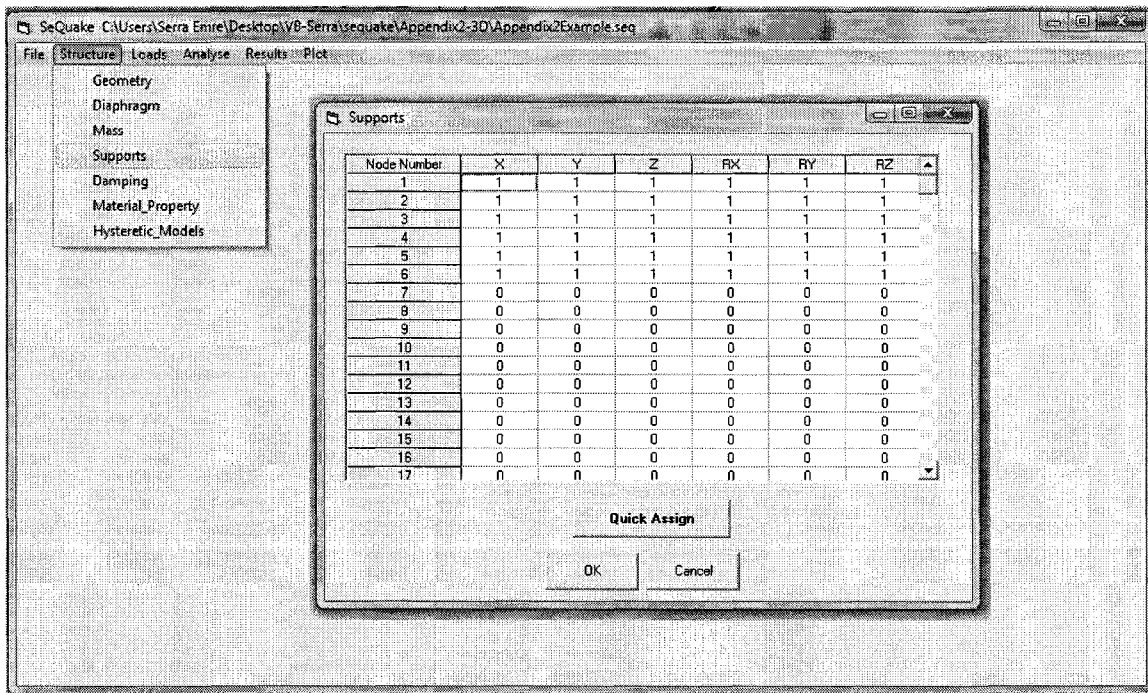


Figure A3.10 Support Information

The form shown in Fig. A3.11 is for alpha and beta components of Raleigh damping.

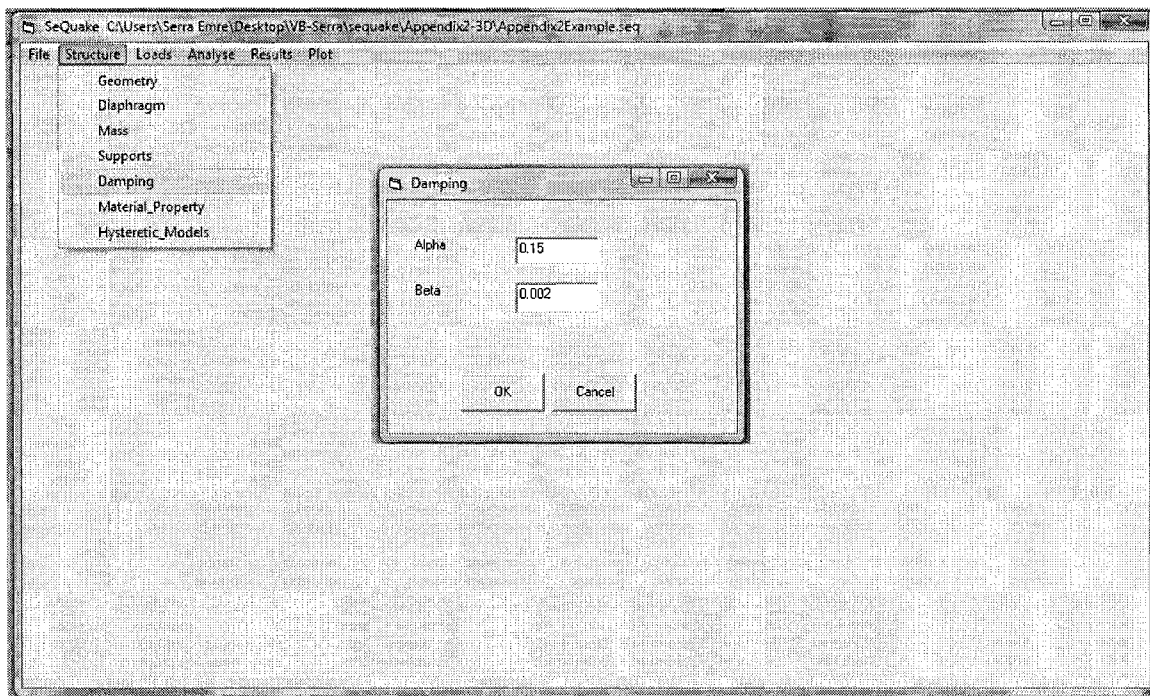


Figure A3.11 Damping Information

In Fig. A3.12, the material types are described which are listed as 1 and 2 in the first column of the first table, the relation between each element and material type can be build with the aid of second table or the use of **Quick Assign** button as shown in Fig. A3.13.

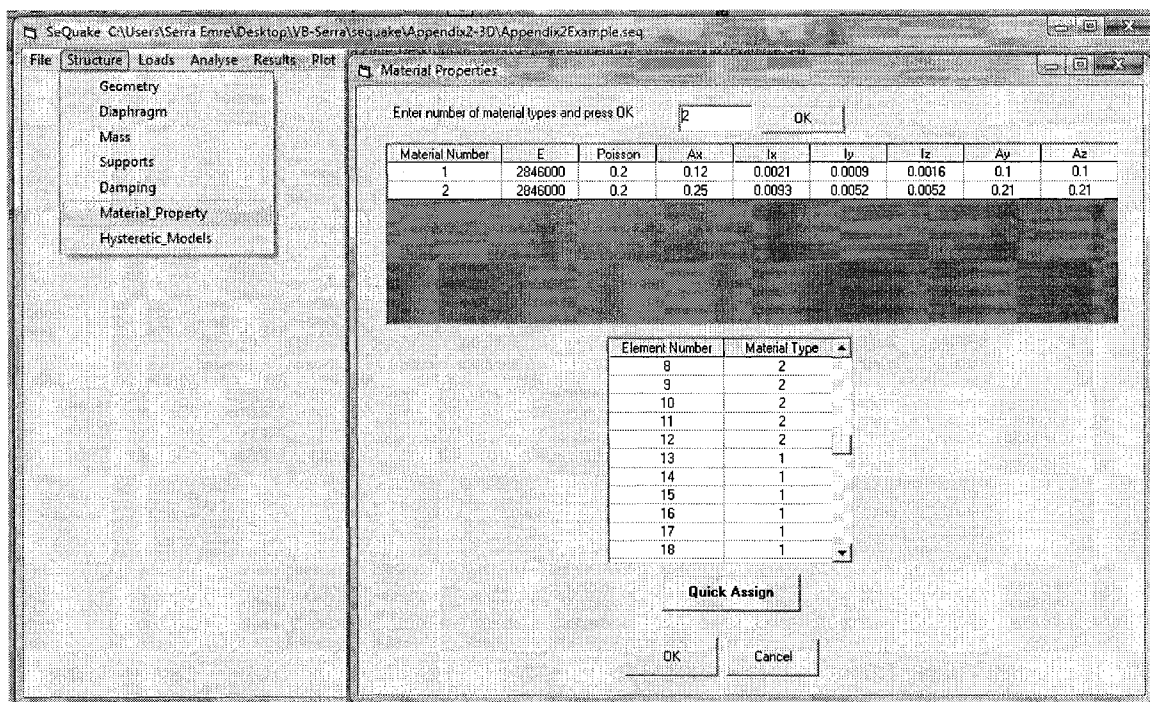


Figure A3.12 Material Information

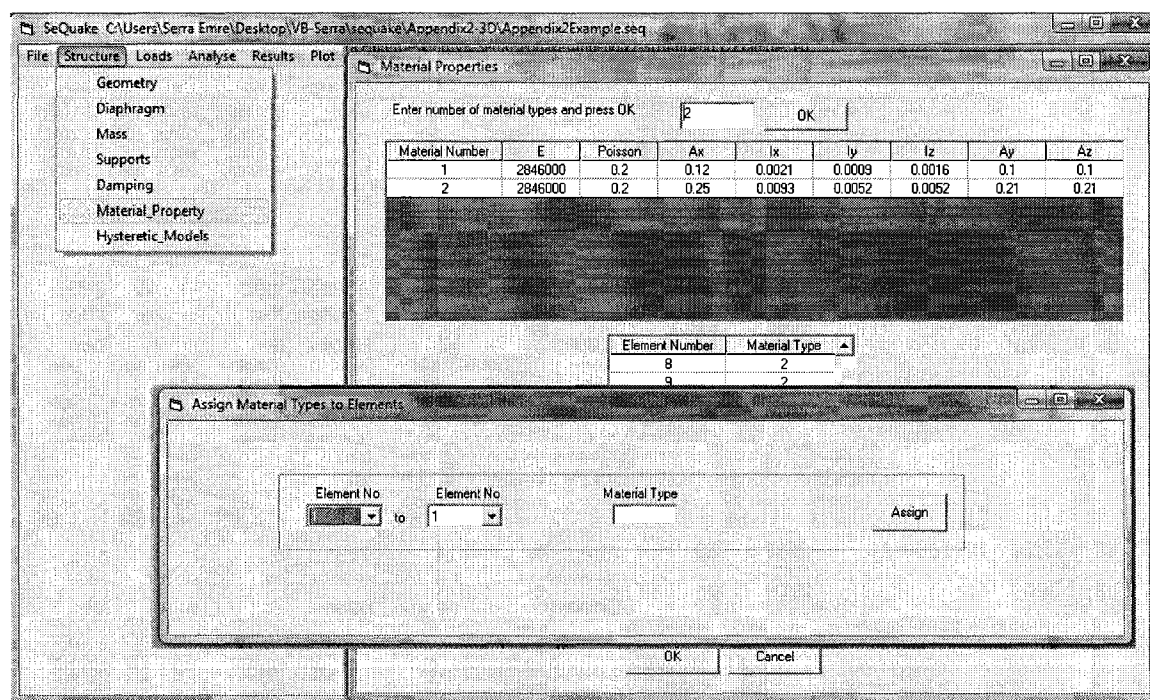


Figure A3.13 Quick Assign for Material Types

In hysteretic model form (Fig. A3.14), the model types are described and the relation between each element and model type can be built by using **Quick Assign** button as shown in Fig. A3.15. The buttons with the question mark sign (?) describe the models as shown in Figs. A3.16 and A3.17.

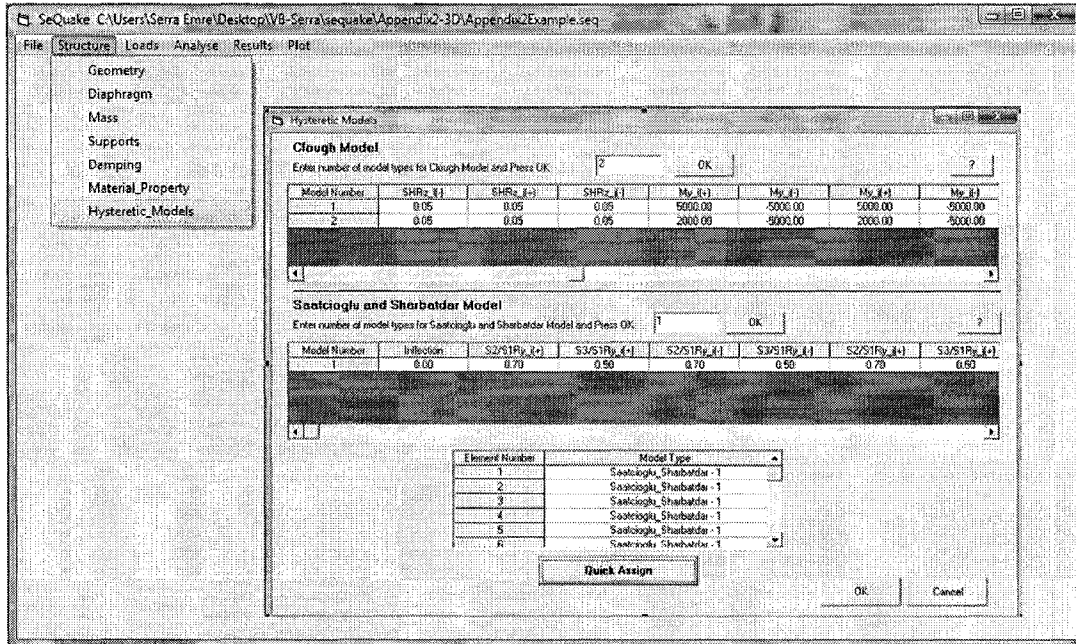


Figure A3.14 Hysteretic Model Information

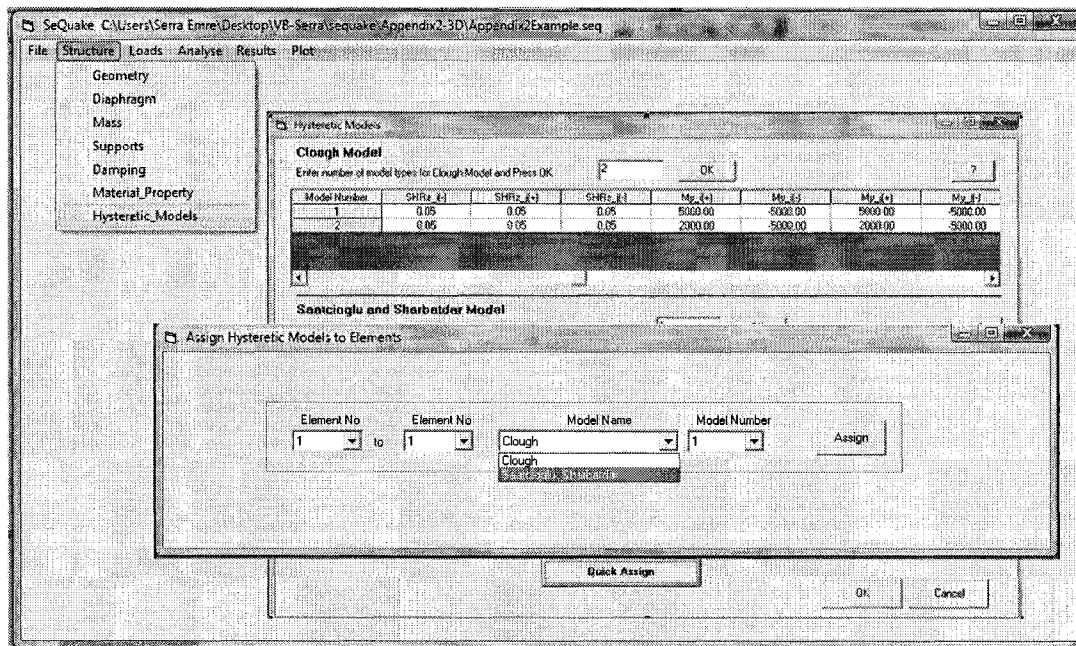


Figure A3.15 Quick Assign for Hysteretic Model Types

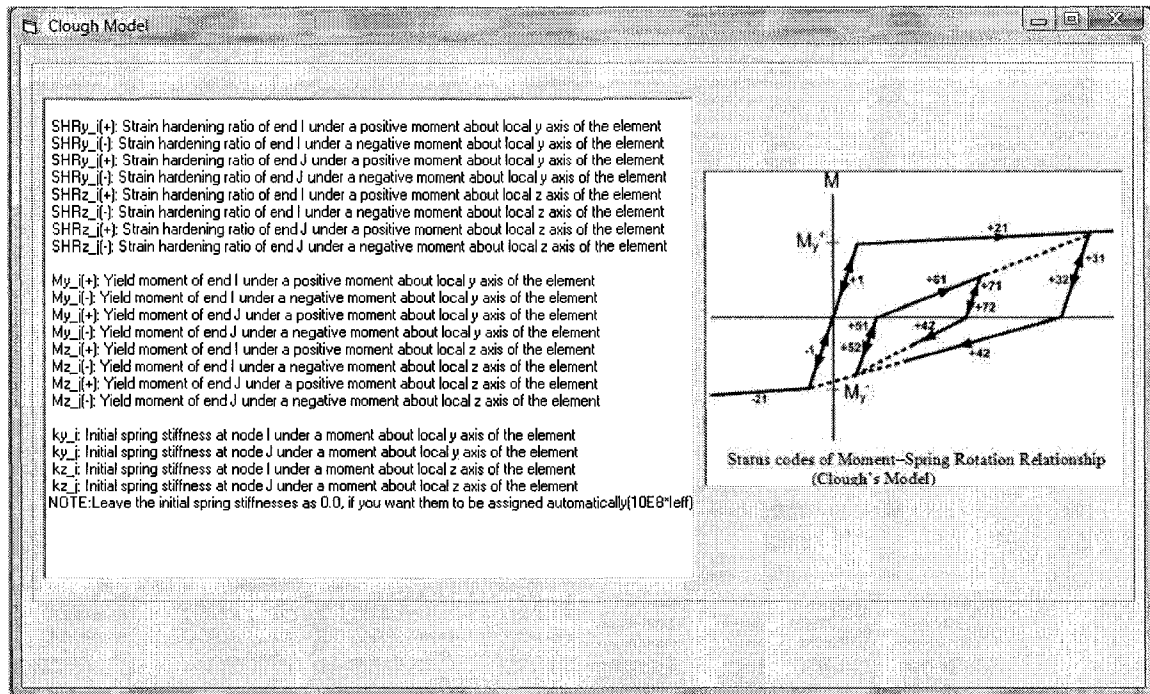


Figure A3.16 Clough's Model

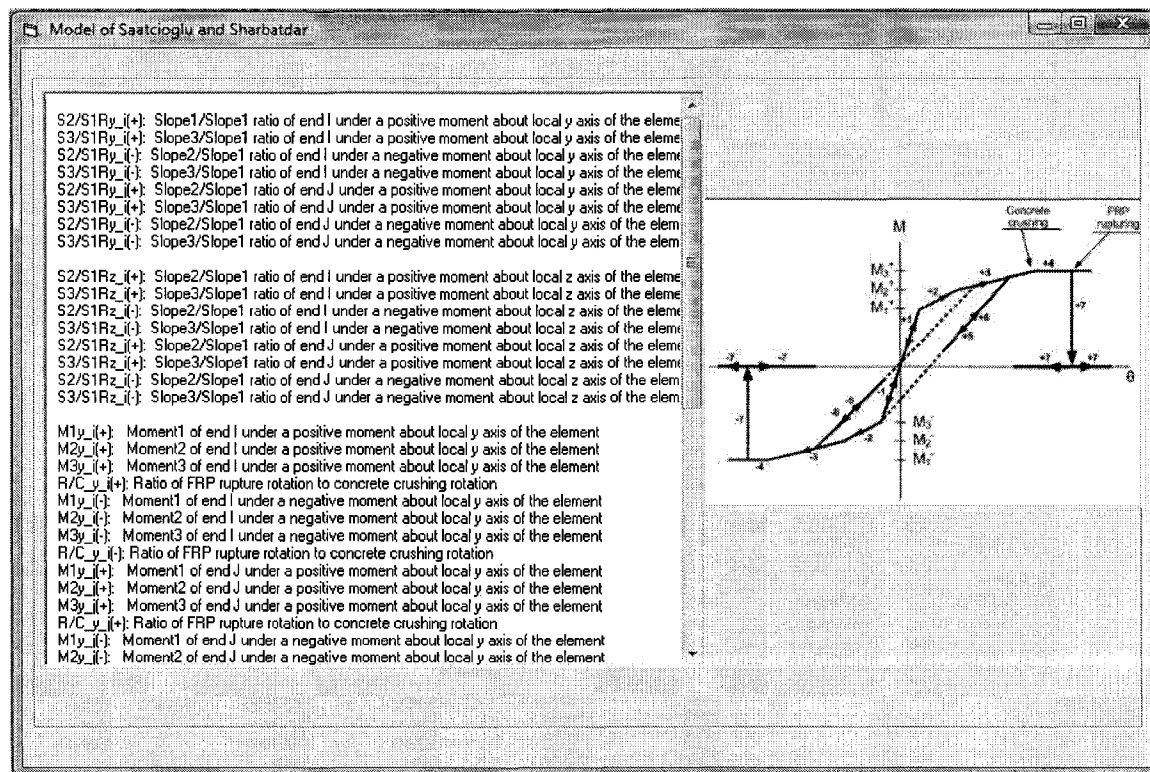


Figure A3.17 Model of Saatcioglu and Sharbatdar

Static Load information can be given for nodal and distributed element loads as shown in Fig. A3.18. The button with the question mark sign (?) explains the load shapes as shown in Figs. A3.19.

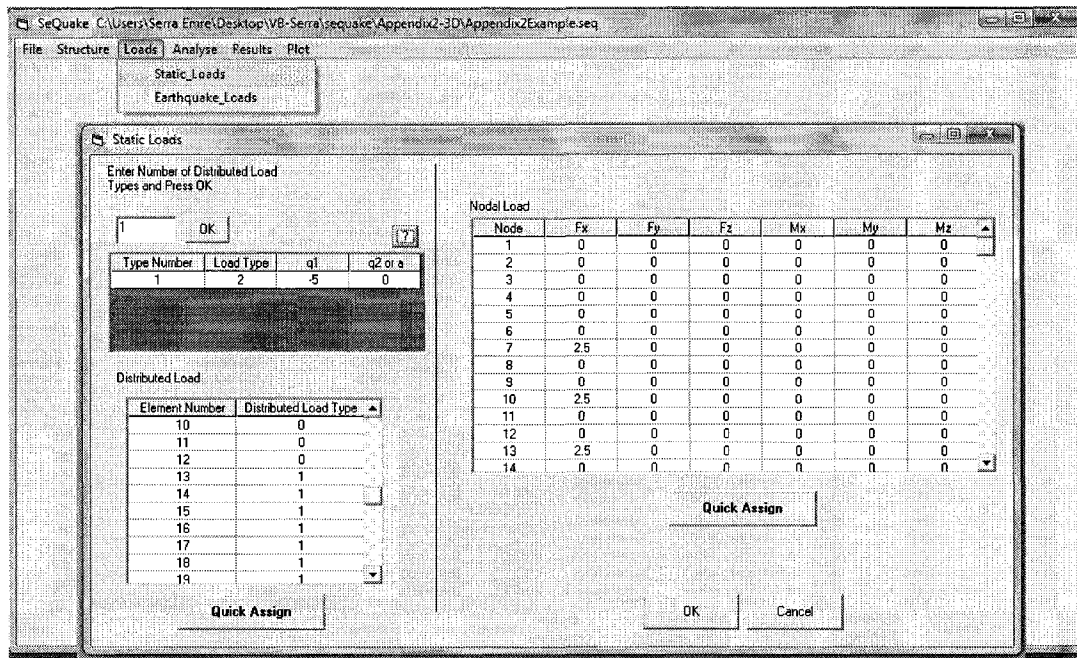


Figure A3.18 Static Load Information

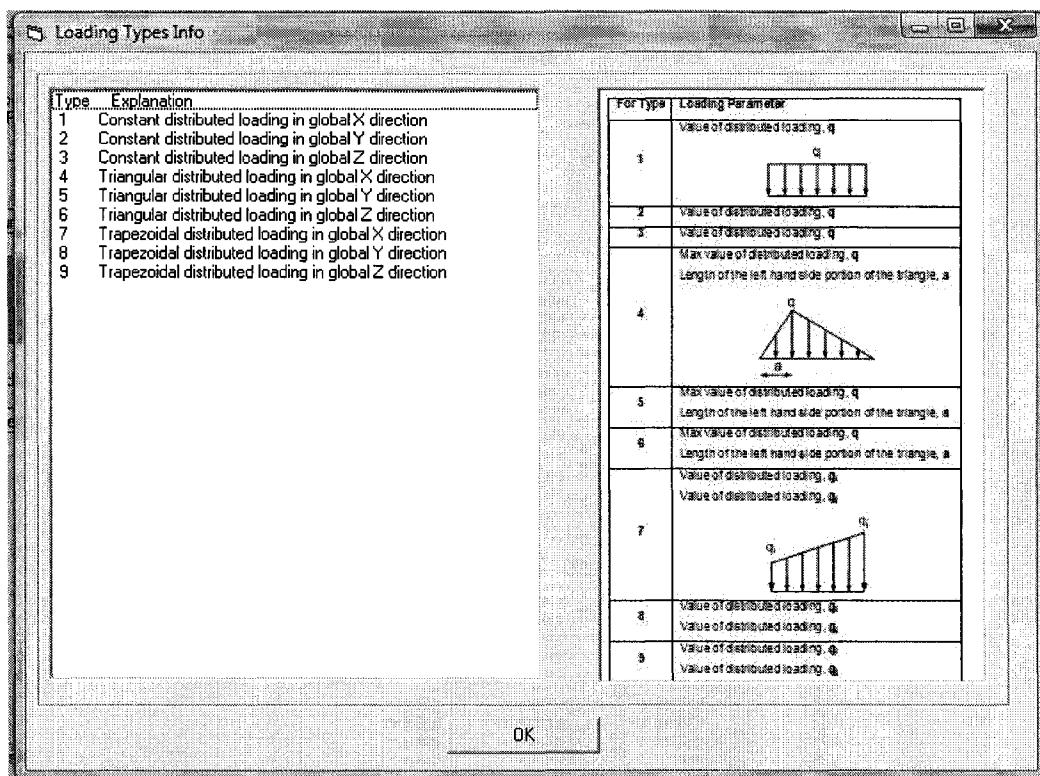


Figure A3.19 Distributed Load Types

As much as ten earthquake record data can be imported as shown in Fig. A3.20. An imported record data can be replaced by another one by choosing it from the list below the **Import EQ File** button and then importing the new one. It should be noted that **Clear** button is for deleting all imported earthquake information.

Importing several records does not mean that the building will be analyzed with one or all of them; the selection of the record(s) for the analysis and all related information will be assigned later as described in the following pages.

Also, the earthquake record file should include only the acceleration information, but not the time.

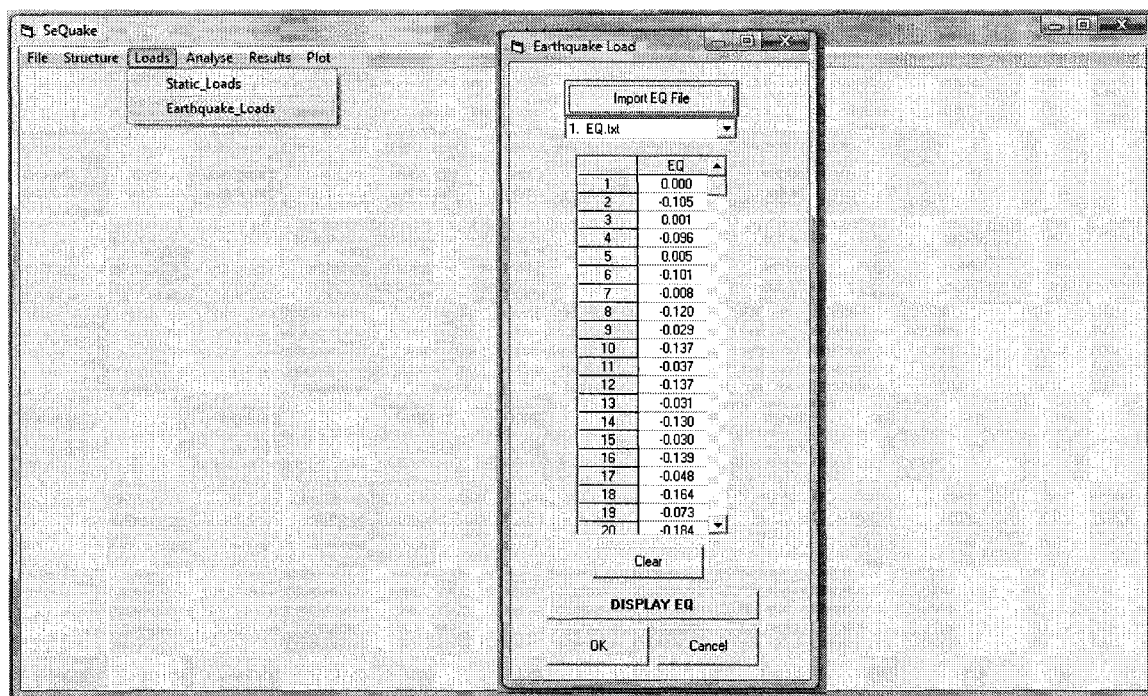


Figure A3.20 Importing EQ records

DISPLAY EQ button opens the form shown in Fig. A3.21. **Show EQ** button in this form plots the earthquake record.

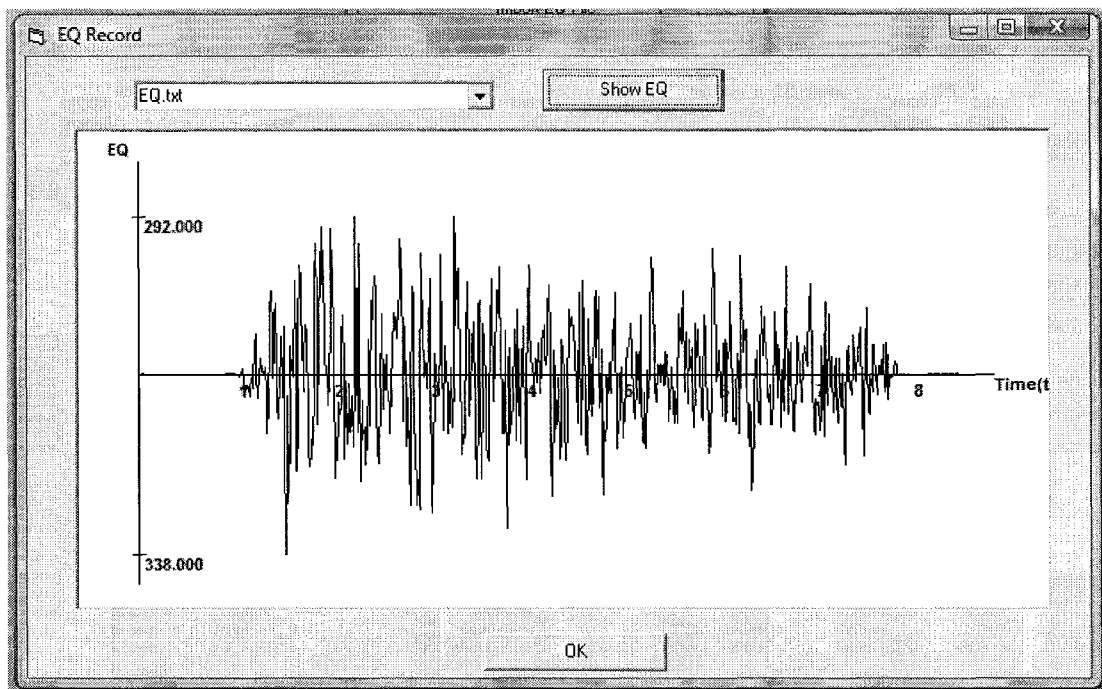


Figure A3.21 Display EQ records

SEQUAKE performs three types of analyses: i) Static and Eigenvalue analysis, ii) Linear Time History analysis and iii) Nonlinear Time History analysis. The window shown in Fig. A3.22 appears after Static and Eigenvalue analysis is performed.

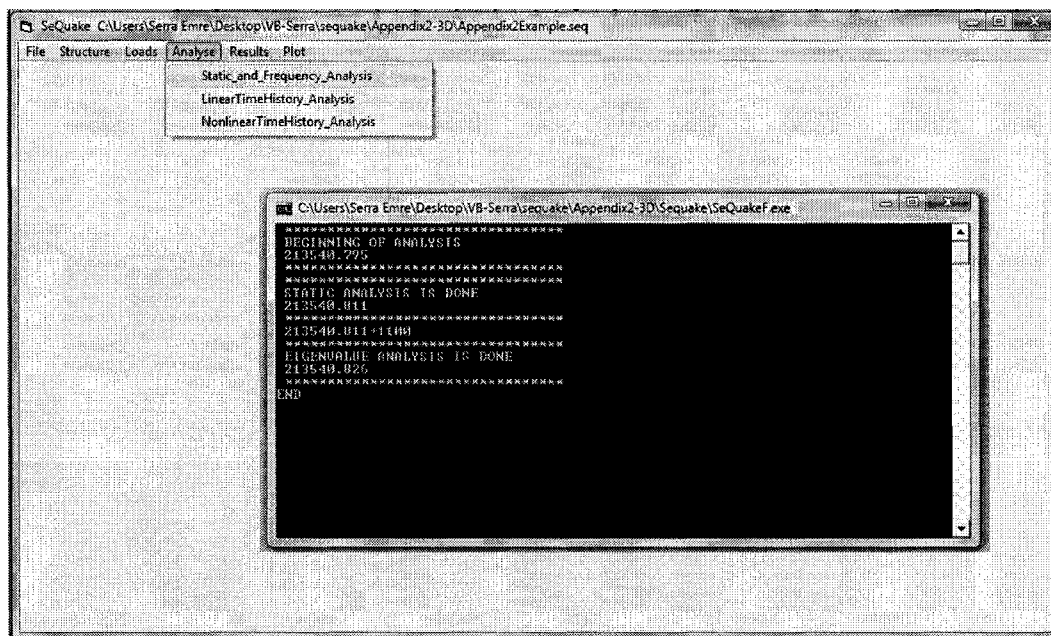


Figure A3.22 Static and Eigenvalue Analysis

In order to perform Linear or Nonlinear Time History (Fig. A3.23) analyses the earthquake record(s) and their directions should be selected. Time interval and multiplication factor for the record(s) as well as Newmark constants are also entered in this form. **OK** button saves the information.

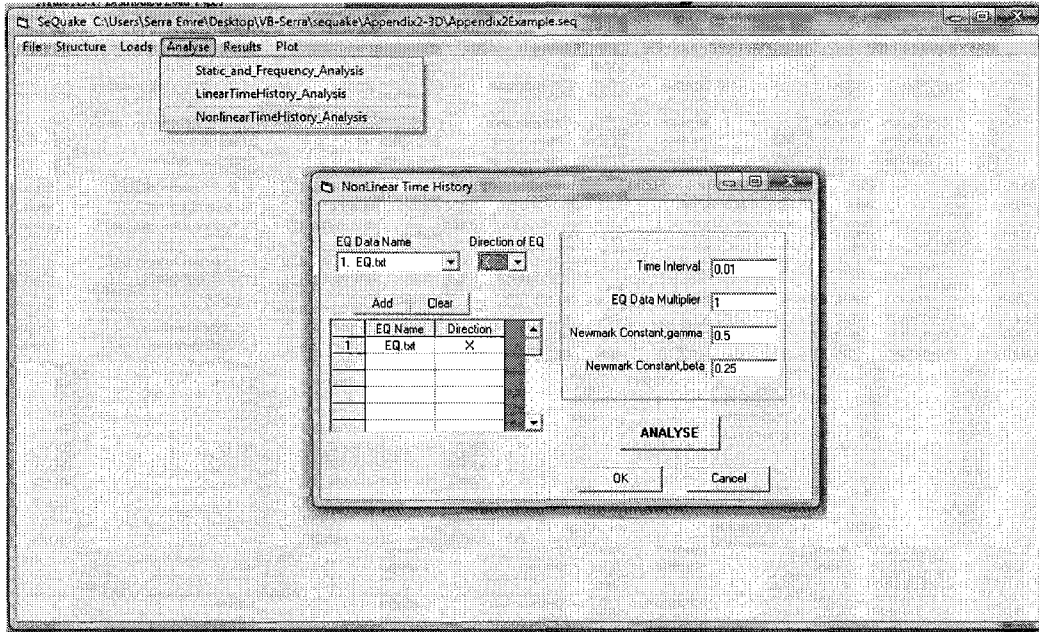


Figure A3.23 Information related to the earthquake records

As a result of the analysis the response history of the selected nodes and elements are printed. These selections will be made after in the form shown in Fig. A3.24 which will appear after **ANALYSE** button is clicked. For selected nodes the displacement histories and for selected elements the moment-chord rotation responses will be printed. Clicking on the **OK** button in Fig. A3.24 starts the analysis and a window is opened as shown in Fig. A3.25.

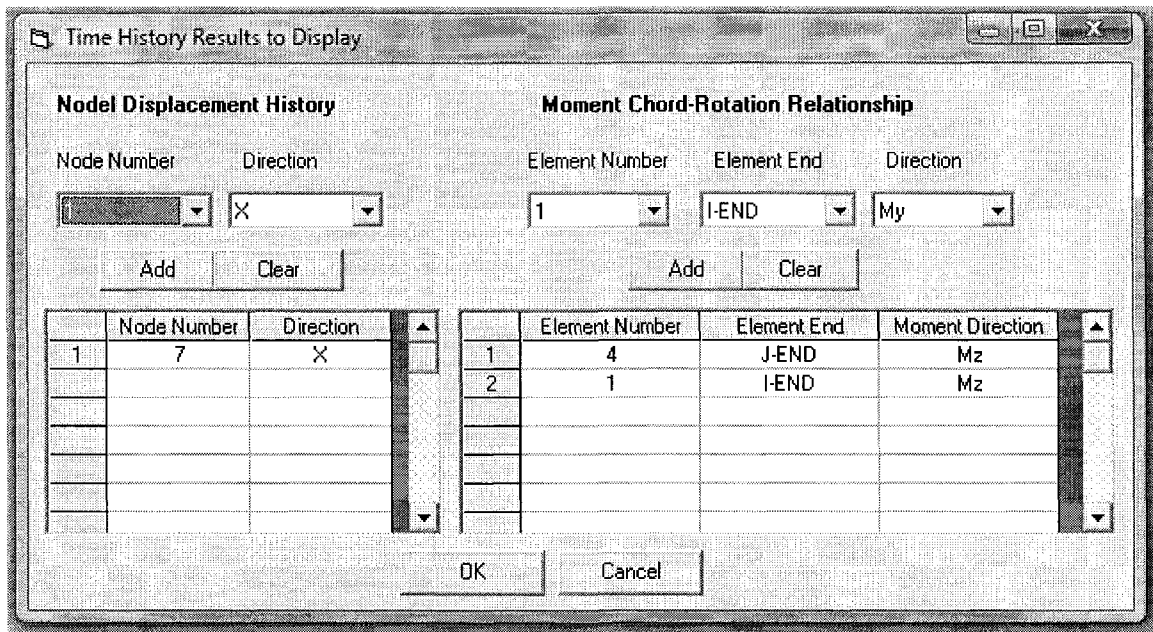


Figure A3.24 Selection of history results to display

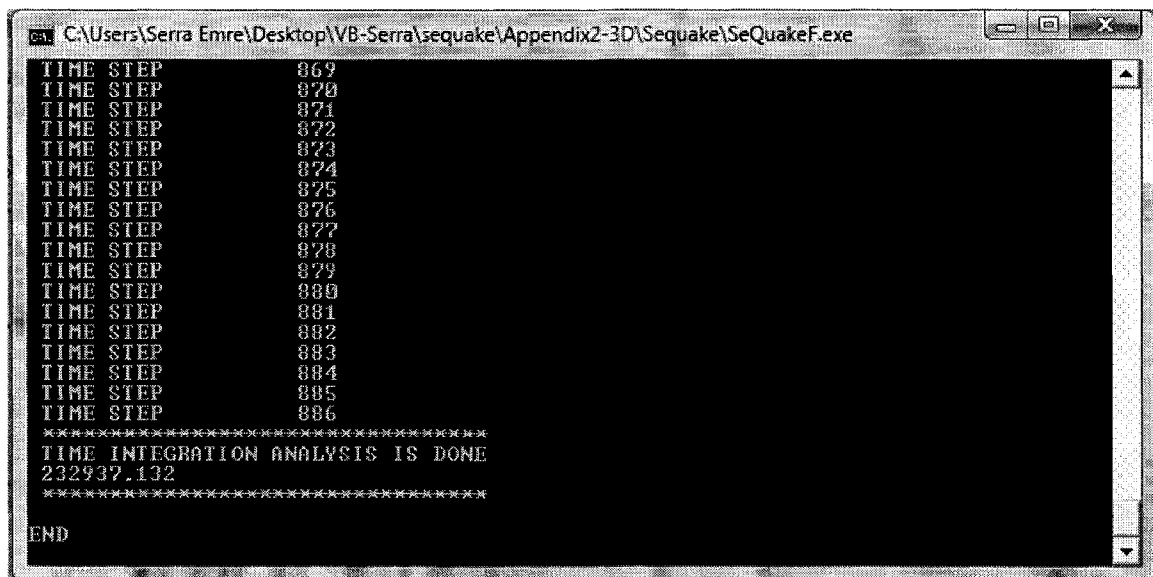


Figure A3.25 Time History Analysis

All of the result files described in Appendix 2 can be displayed from Results Menu (Fig. A3.26) in text form. For instance clicking on Eigenvalue will display EIGENVALUE.SEQ as shown in Fig. A3.27. **DeleteResults** button deletes all the results.

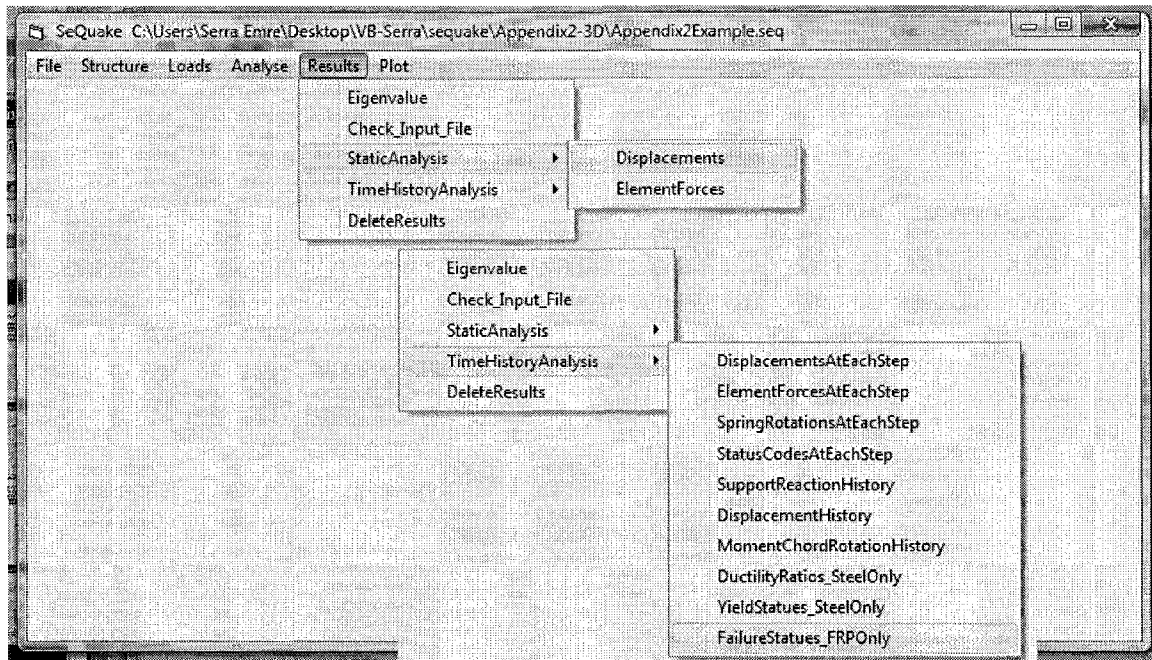


Figure A3.26 Result Menu

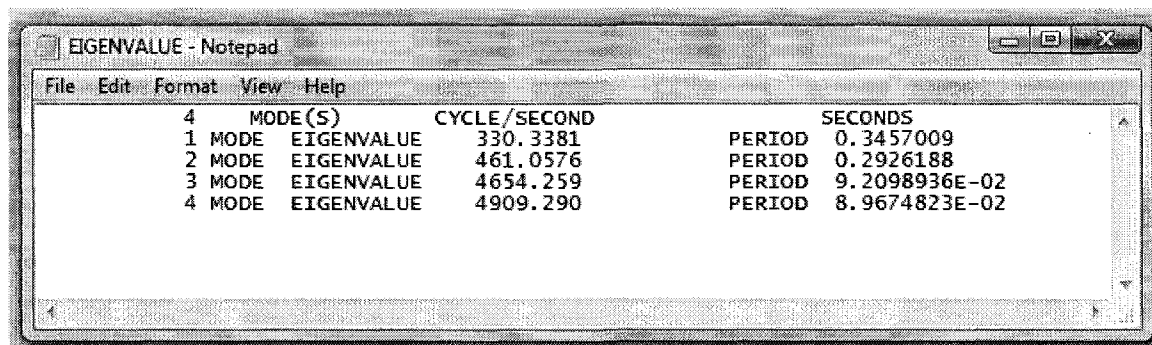


Figure A3.27 EIGENVALUE.SEQ

Plot Menu shown in Fig. A3.28 is for displaying the structure geometry as well as the support reaction, displacement and moment chord rotation histories (Figs. A3.29-33). OpenGL window shows the general geometry of the structure with mass, support, static loading information and node/element numbers.

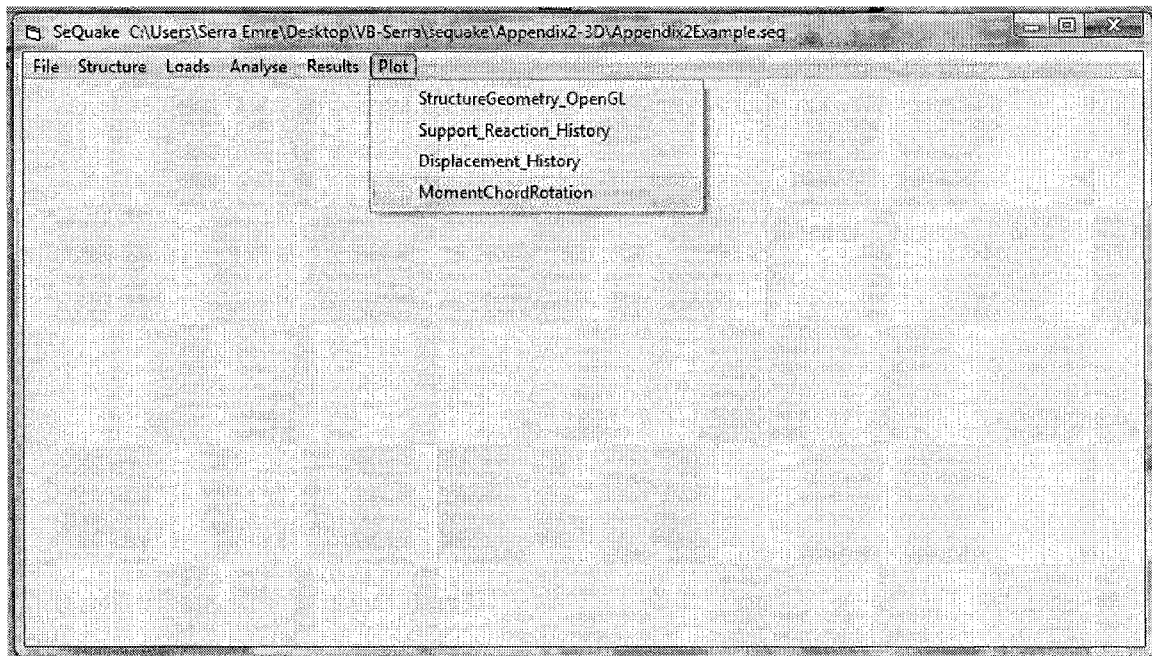


Figure A3.28 Plot Menu

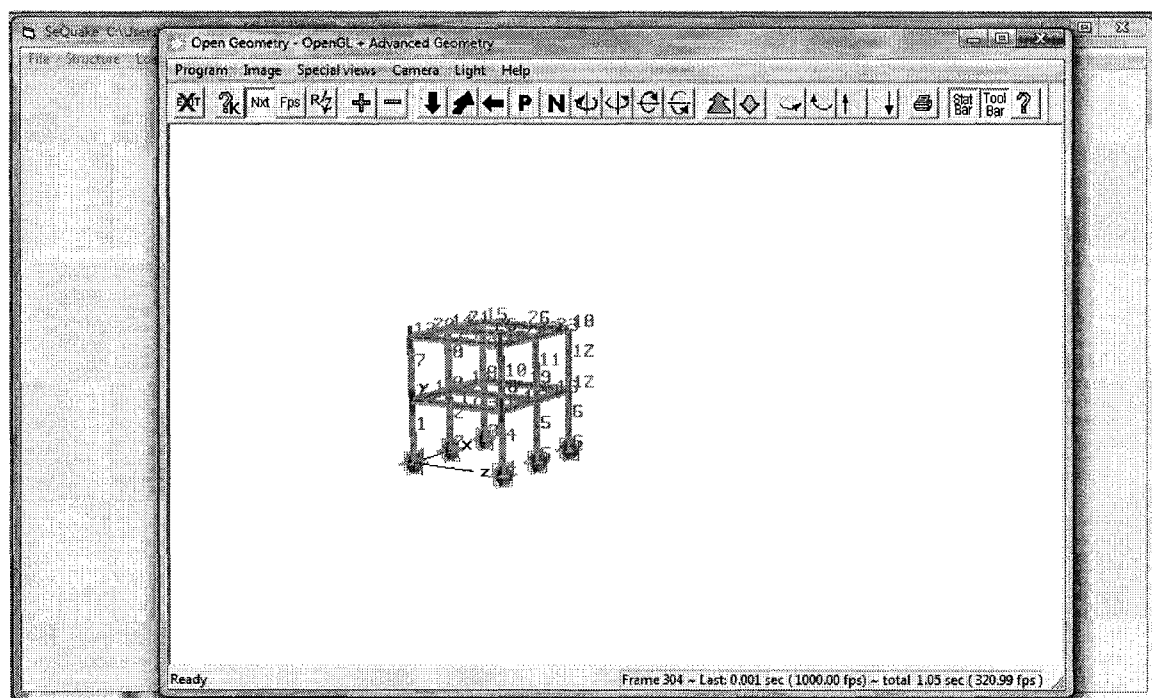


Figure A3.29 Structure geometry with OpenGL

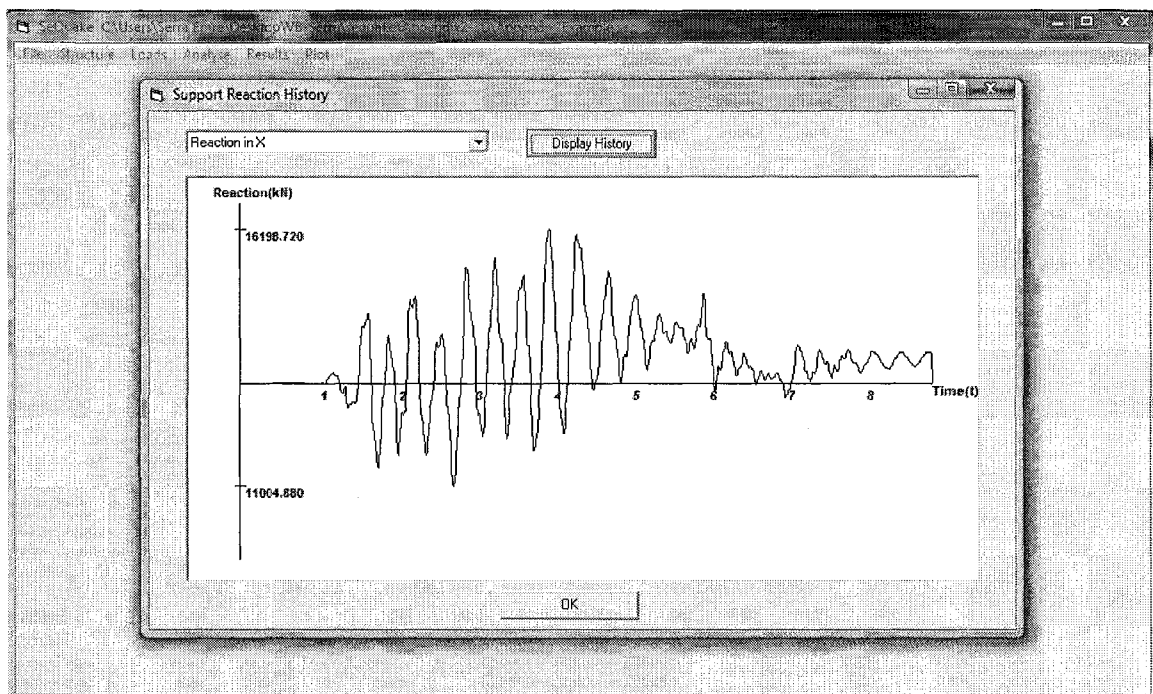


Figure A3.30 Support reaction history

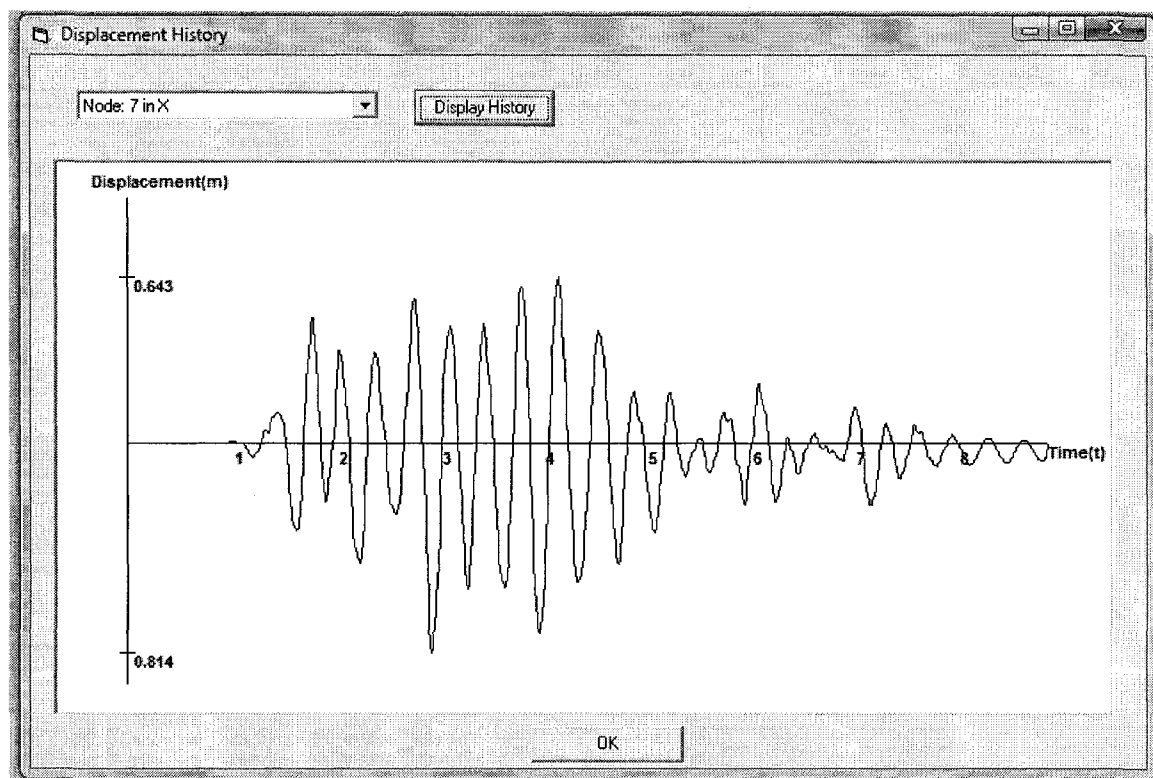


Figure A3.31 Nodal displacement history

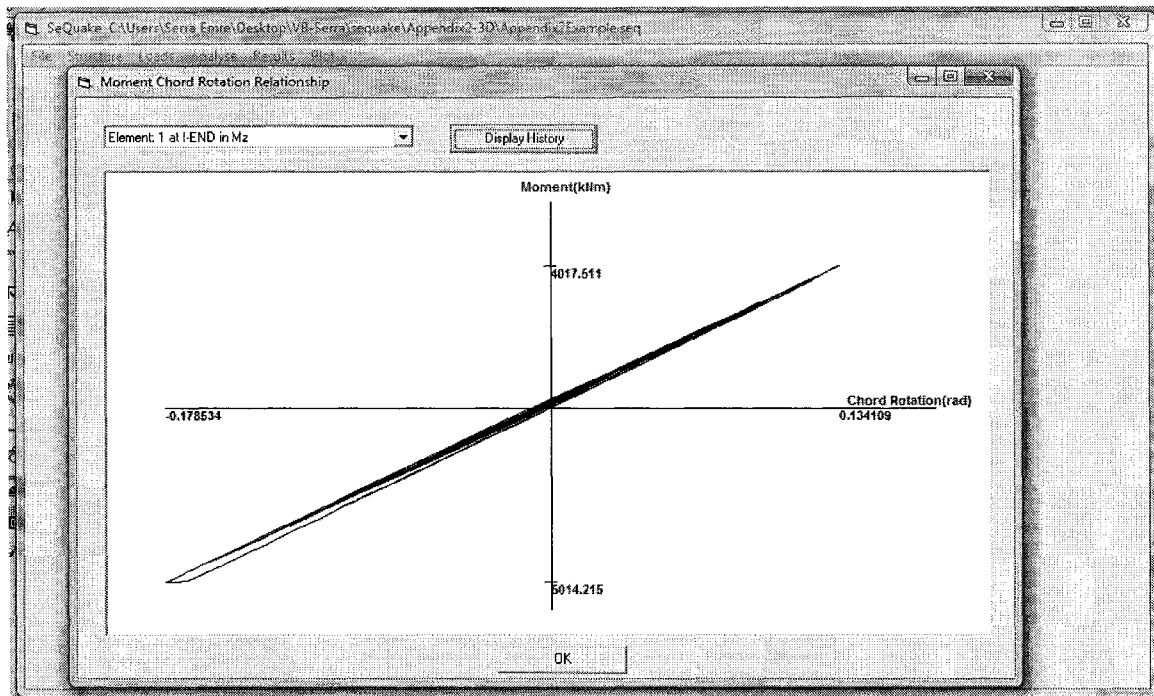


Figure A3.32 Moment-chord rotation relationship for a steel reinforced member

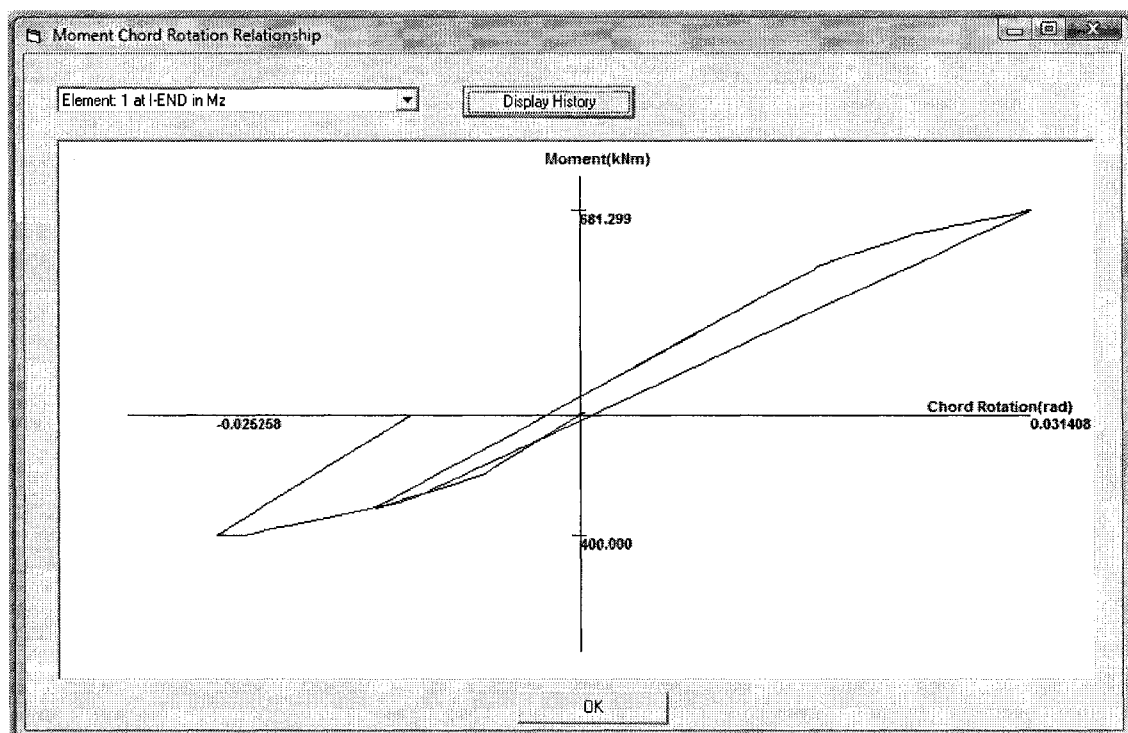


Figure A3.33 Moment-chord rotation relationship for a FRP reinforced member

APPENDIX 4

Data Used for Design of Buildings

Table A4.1 Loads and material properties

Dead Loads (kN/m²) (including beams and columns)	<i>Floor</i>	5
	<i>Roof</i>	3.5
Live Loads (kN/m²) (including beams and columns)	<i>Floor</i>	2.4
	<i>Roof</i>	2.2
Concrete Compressive Strength, f'_c (MPa)	40	
Modulus of Elasticity of Concrete, E_c (MPa)	28460	
Steel Yield Stress (MPa)	400	
Modulus of Elasticity of Steel, E_s (MPa)	200000	
CFRP Tensile Stress (MPa)	1596	
Modulus of Elasticity of CFRP, E_{FRP} (MPa)	111100	

Table A4.2 Weight at each floor for 5, 10 and 15 storey buildings

Roof (kN)	3348
Floor (kN)	2079

Table A4.3 Static lateral loads for 5 storey RC buildings

	Steel RC Building		FRP RC Building	
Storey	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)
5	135	172	458	1173
4	173	222	590	1511
3	130	167	443	1133
2	87	111	295	755
1	43	56	148	378

Table A4.4 Static lateral loads for 10 storey RC buildings

	Steel RC Building		FRP RC Building	
Storey	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)
10	71	121	242	797
9	103	175	351	1155
8	92	155	312	1026
7	80	136	273	898
6	69	117	234	770
5	57	97	195	641
4	46	78	156	513
3	34	58	117	385
2	23	39	78	257
1	11	19	39	128

Table A4.5 Static lateral loads for 15 storey RC buildings

	Steel RC Building		FRP RC Building	
Storey	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)	Ottawa EQ Loads(kN)	Vancouver EQ Loads(kN)
15	52	100	176	678
14	78	150	264	1019
13	72	139	245	946
12	67	128	226	873
11	61	118	207	801
10	56	107	189	728
9	50	96	170	655
8	44	86	151	582
7	39	75	132	509
6	33	64	113	437
5	28	54	94	364
4	22	43	75	291
3	17	32	57	218
2	11	21	38	146
1	6	11	19	73

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