Effects of Strain Rate Dependency on Moment Resisting Frames with
Reduced Beam Sections

Submitted by

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A thesis submitted to the Faculty of Graduate Studies and Postdoctoral
Affairs in partial fulfillment of the requirements for the degree of

Master of Applied Science

In

Civil Engineering

Carleton University

Ottawa, Ontario

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ABSTRACT

There have been numerous numerical and experimental studies conducted to investigate the performance of a moment frame with reduced beam section (RBS). However, most of the numerical studies have not been carried out using realistic material properties and constitutive model that can more accurately represent the material response. Strain rate dependent material properties and parameters for mixed-mode hardening model for two heats of steel have been considered. One heat has the yield strength close to the minimum yield strength of CAN/CSA G40.20/21 350W steel and the other has the yield strength close to the probable yield strength of 385 MPa for grade 350W steel. Non-linear dynamic analyses on four-storey moderately ductile and limited ductility moment resisting frames with RBS for a suite of scaled ground motions have been carried out to investigate effects of material properties strain rate dependency on the response of the frame.

Strain rate and deflection (inter-storey drift) have been found to be independent of material strength and material properties strain rate dependency. However, the strain rate is high enough that effects of material properties strain rate dependency on the structural response can be important. The mean increase in the moment at the center of RBS due to material properties strain rate dependency can be as high as 10%, and the mean predicted maximum moment can be greater than the calculated probable moment at the center of RBS by up to 13%. With strain rate dependent material properties, the mean predicted maximum base shear can exceed the design base shear by up to 20% and the predicted mean strain hardening factor calculated with respect to the initial static yield stress can be higher than the prescribed factors by up to 25%.

ACKNOWLEDGEMENT

I would like to thank my thesis supervisor Dr. Heng Aik Khoo for his continuous support, persistence, and guidance throughout my research program. I would like to express my utmost gratitude to him for patiently teaching and advising me in every problem I faced in my research. His contribution in this research has been invaluable. I got very honest and valuable feedbacks from him which drove me to give my best. I thank him for giving me research assistantship to fund my research program.

I immensely thank Mr. Cuckoo Kochar for providing me with Kochar Family Scholarship which gave me financial support in my master's program. I would also like to thank the department of Civil and Environmental Engineering at Carleton University for employing me as a teaching assistant.

I would like to thank my thesis committee members, Dr. Magdi Mohareb (University of Ottawa) and Dr. Abhijit Sarkar (Carleton University) for their insightful comments.

I would like to extend my thanks to Mr. Tarundeep Singh for guiding me and sharing experiences from his own research. His advice gave direction to my research work.

I would like to thank my mother Mrs. Reshma Joshi and my father Mr. Yogesh Joshi for giving me strength, support, and courage to pursue my dreams. This would not have been possible without their immense love, care and understanding. I would also like to thank my sister Kalyani for her constant encouragement and belief in me. I would like to thank Mahavir for listening to me and supporting me throughout the journey of my graduate studies.

Finally, I would like to show my immense gratitude to the almighty lord Ganesh for giving me abundant courage, knowledge, and power to fight and succeed against all odds.

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LIST OF SYMBOLS AND ABBREVIATIONS

a	-	transition length of the reduced beam section
A	-	modified amplification equation constant
AISC	-	American Institute of Steel Construction
ASTM	-	American Society for Testing and Materials
ATC	-	Applied Technology Council
B	-	modified amplification equation constant
B31	-	linear 3D beam
B31OS	-	linear 3D beam element with open section
BFP	-	bolted flange plate connection
BSEP	-	bolted stiffened end plate connection
BUEP	-	bolted unstiffened end plate connection
$b(z)$	-	beam flange width along the longitudinal axis
b_c	-	width of flange
c	-	depth of the cut at the center of reduced beam section
C_e	-	design Pseudo acceleration
$[C]$	-	damping matrix for multiple degree of freedom system
CISC	-	Canadian Institute of Steel Construction
CSA	-	Canadian Standard Association
CUREE	-	Consortium of Universities for Research in Earthquake Engineering
D_a	-	isotropic yield stress amplification factor
d_c	-	depth of column
d_b	-	depth of beam

F_A	-	fundamental time period
F_{sh}	-	strain hardening factor
F_y	-	tensile yield strength
FEMA	-	Federal Emergency Management Agency
G	-	shear modulus
GMPEs	-	ground motion prediction equations
I	-	Cowper-Symonds equation constant
I_E	-	importance factor of a structure
J	-	Cowper-Symonds equation constant
$[K]$	-	stiffness matrix for multiple DOF system
K_n	-	generalized stiffness system for damping
l_p	-	equivalent length of plastic hinge
$L(t)$	-	length at any time t
LD1	-	limited ductility moment resisting frame designed to meet both inter-storey drift limit and strength criteria
$[M]$	-	mass matrix
M_n	-	generalized mass system for damping
M_{pr}	-	maximum probable moment at center of RBS
M_{cf}	-	maximum probable moment at face of the column
M_c	-	maximum probable moment at centerline of the column
MD1	-	moderately ductile moment resisting frame designed to meet the inter-storey drift limit criteria

MD2	-	moderately ductile moment resisting frame designed to meet the strength criteria
MPC	-	multi-point constraint
MRF	-	moment resisting frame
n	-	number of kinematic hardening terms
NBCC	-	National Building Code of Canada
NEHRP	-	National Earthquake Hazards Reduction Program
PGA	-	peak ground acceleration
PGV	-	peak ground velocity
PM1	-	preliminary model with maximum RBS flange width reduction of 46%
PM2	-	preliminary model with maximum RBS flange width reduction of 32%
RBS	-	reduced beam section connection
RSN	-	record sequence number
R	-	seismic force reduction factor
R_d	-	ductility related force modification factor
R_o	-	overstrength- related modification factor
R_y	-	ratio of probable yield stress to minimum yield stress
s	-	length of reduced beam section
SEAOC	-	Structural Engineers Association of California
S4R	-	4-noded shell element with reduced integration
$S(a)_T$	-	spectral acceleration at fundamental time period

$S(a)_{median}$	-	mean spectral acceleration
$S(T)$	-	design spectrum
$S_T(T)$	-	target response spectrum
K_0	-	parameter defining the size of yield surface
T_R	-	time period range
T_{RS}	-	scenario-specific time period range
C_n	-	kinematic hardening moduli
$\dot{\epsilon}$	-	strain rate in s^{-1}
θ_p	-	plastic hinge rotation at ultimate load
ϕ_u	-	curvature of beam at ultimate load
ϕ_y	-	curvature of beam at yielding load
σ^i	-	isotropic yield stress
$\sigma^i(\dot{\epsilon})$	-	isotropic yield stress at a strain rate
σ_s^i	-	isotropic yield stress at zero strain rate
σ_t^F	-	true flow stress at zero strain rate
$\sigma_t^F(\dot{\epsilon})$	-	true flow stress at a strain rate
σ_y	-	static yield stress
σ_s^{yp}	-	isotropic yield stress at the end of yield plateau
σ_{eq}^i	-	equivalent isotropic yield stress
α	-	back stress/Rayleigh damping coefficient
α_{f_j}	-	scaling factor of individual ground motion
β	-	Rayleigh damping coefficient

β_g	-	scaling factor for the entire suite of earthquake records
γ_n	-	rate of C_n reduction with increasing plastic strain
ε_t^p	-	true plastic strain
ε_0^p	-	true plastic strain at the end of yield plateau
$d\varepsilon^{pl}$	-	equivalent plastic strain rate
$\dot{\varepsilon}_c$	-	critical strain rate
ε	-	damping ratio
s_{ij}	-	deviatoric stress tensor
$S(\dot{\varepsilon})$	-	isotropic yield stress amplification factor at infinite plastic strain
V_0	-	activation volume constant
k_b	-	Boltzmann constant
f_{ys}	-	static yield strength
f_{yd}	-	dynamic yield strength
ω_i	-	frequency for the mode considered in damping

Chapter 1: Introduction

In the 1994 Northridge earthquake, severe damages were caused by the high energy dissipation in welded beam-column connections of many steel moment frames that resulted in the brittle failure of the connections. This spurred extensive studies into developing structural elements which could avoid the failure of beam-column connections during a seismic event. Design recommendations for these structural elements through extensive research funded by Federal Emergency Management Agency (FEMA) were published in the Seismic Provisions of Structural Steel Building by American Institute of Steel Construction (AISC) and FEMA 350 (2000). Recommendations in FEMA 350 includes detailing that induces plastic hinge for energy dissipation at a short distance away from the beam-column connection. Reduced beam section (RBS) connection is one of the methods described in FEMA 350 to artificially induce plastic hinge formation away from the beam-column connection, and thus avoiding the beam-column connection failure.

A seismic event can produce a strain rate of up to 1.0s^{-1} in a component of a steel structure (Tian et al. 2014). This strain rate is much higher than the quasi-static rate with which material properties used in steel structures design are determined. Most of the numerical studies were also based on quasi-static material properties. Thus, proper representation of the material behavior is necessary to predict a more accurate response of the structure in seismic analyses. With the use to strain rate dependent material properties, a more realistic response of the steel structure can be predicted from the seismic analyses.

There have been many numerical analyses and experimental testing conducted to determine the behavior of beam-column connection with RBS under monotonic and cyclic loading. However, very few studies have been performed to investigate the non-linear

dynamic behavior of RBS in a steel moment resisting frame (MRF) under seismic loading using a plasticity model with realistic material properties. In addition, no analysis has been carried out to study the non-linear behavior of moderately ductile and limited ductility MRFs with RBS under earthquake loading that considered material properties strain rate dependency.

1.1 Objective of the thesis

The objective of the thesis is to understand the response and performance of a moderately ductile and limited ductility MRFs with RBS connections subjected to seismic loading. Numerical simulations conducted for these MRFs consider strain rate material properties dependency in order to predict a more accurate structural response. Comparisons of the results are made to investigate effects of material properties strain rate dependency on the structural response and on the results with respect to the design provisions.

1.2 Methodology used in the research

Singh (2019) has calibrated strain rate dependent material properties and parameters for the mixed-mode hardening plasticity model by Lemaitre and Chaboche (1990) for one heat each of CAN/CSA G40.20/21 300W and ASTM A572 grade 50 steel. Material properties and parameters for these two heats of steel are used to conduct the non-linear dynamic time history analyses in order to predict a more realistic structural response. These material properties and parameters, and modelling of RBS are validated against cyclic loading experimental test results of a deep column-beam connection with RBS by Chi and Uang (2002) in numerical simulations under quasi-static condition.

A four-storey building with either moderately ductile or limited ductility MRFs with RBS connections located at Powell River, British Columbia on a site class D are considered in this study. These MRFs with RBS connections are designed according to Moment Connection for Seismic Application (CISC 2014), National Building Code of Canada (NBCC 2015) and CSA S16-14 (CSA 2014). Two moderately ductile MRFs are considered with one frame designed to meet the strength and inter-storey drift limit criteria and other only the strength criteria. One limited ductility MRF is designed to meet both inter-storey drift and strength criteria. Single storey moderately ductile MRFs with RBS connections with two different ratios of the maximum cut of RBS flange width are first designed and analysed in order to conduct mesh sensitivity study to select the mesh (element) sizes and also to select the ratio of maximum cut to be used in the four-storey building analyses.

All numerical analyses are carried out with the finite element analyses software package ABAQUS (ABAQUS 2017). Numerical simulations of non-linear dynamic time history analyses for these three MRFs with strain rate and non-strain rate (static) dependent material properties subjected to a suite of seismic records are performed. The Method-A spectral matching scaling technique recommended by National Building Code of Canada 2015 (NBCC 2015) is used to scale the seismic records. Results from numerical analyses on strain rate, bending moment, inter-storey drift, base shear and strain hardening are analysed and compared to study effects of strain rate dependent material properties on the structural response and against the design provisions.

1.3 Organization of thesis

Chapter 2 presents a brief literature review of the historical background, capacity design criteria, experimental and analytical studies on moment resisting frames (MRFs) with reduced beam section (RBS) connections. A brief discussion on strain rate dependency of material properties, plasticity constitutive model and effects of strain rate on structures under seismic loading.

Chapter 3 presents the calibrated parameters from previous studies that are used in generating the material properties (input) for the mixed-mode hardening plasticity model used in this study. Design and numerical modelling of single storey moderately ductile MRFs with RBS connections for the preliminary study are provided. Mesh sensitivity analyses are conducted in the preliminary study. The mesh (element) sizes are selected for analyses of the four-storey MRFs with RBS connections. In the preliminary study, the target ratio of the maximum cut of RBS flange width to be used in the 4 storey MRFs is also discussed. Design of the 4 storey MRFs with RBS connections are presented.

Chapter 4 consists of finite element modelling of a column-beam moment connection with RBS corresponding to the cyclic loading test conducted by Chi and Uang (2002). A combination of shell and beam element model (hybrid model), and beam element only model are explored in the modelling. The material properties and parameters are validated against the test results. Comparisons are made between results of hybrid and beam element only models to show the acceptability of using the beam element only model in the analyses.

Chapter 5 presents results from the non-linear dynamic time-history analyses with strain rate and non-strain rate (static) dependent material properties of 4-storey moderately

ductile and limited ductility moment resisting frames with RBS connections subjected to a suite of earthquake records. The method used to scale the earthquake records is discussed. Results for strain rate, bending moment at various critical locations, inter-storey drift, base shear and strain hardening are presented. These results are compared and discussed with respect to effects of strain rate dependency of material properties and against design provisions.

Chapter 6 presents conclusions and recommendations for future studies.

Supplementary numerical analyses and test results are provided in the appendices.

Chapter 2: Literature Review

Steel can be considered to be a good construction material for earthquake-resistant structures due to its strength, ductility, toughness and ability to dissipate a considerable large amount of energy during inelastic deformation. For a steel building, the lateral force resisting systems can consist of either braced frames, moment resisting frames (MRF) or a combination of these two frames. One of these systems is a moment resisting frame with the reduced beam sections (RBS). Brief discussions of history, design, experimental and numerical analysis of MRF and RBS are provided below. In addition, effects of strain rate on material behavior of steel and seismic performance of structures are reviewed.

2.1 Historical background of moment resisting frames

Steel moment resisting frames have been in use since the late 19th century in North America. A steel moment frame is constructed with of a series of beams and columns where the connections are formed rigidly using the combination of bolting and welding, which are considered as moment connections.

In the early 1920s, construction of steel frames consisted of complex built-up members with gusset plates and built-up connections. The members and connections were riveted as well as encased with masonry or concrete for fire protection and to provide partitions for the structure. However, these structures were designed for wind loads and not for seismic loads (Hamburger and Nazir 2003). Structural engineers depended upon stiff and strong unreinforced masonry and partition walls for the resistance of lateral loads, but calculations for the stiffness and resistance provided by these walls were not performed (Roeder 2000). Over the span of next 20 to 30 years, gusset plates used for connections

were replaced by rivet angles and T-angles. In 1950s, rivets were replaced with high strength bolts. Gradually with time, in the early 1970's, field welding began to replace field bolting (Hamburger and Nazir 2003). Around the same time, a series of tests suggested that connections with welded flanges and high strength bolted webs can achieve inelastic rotation prior to the fracture of the flange welds (Popov and Stephen 1970). The performance of welded steel moment frame in the experimental and analytical investigations, and as well as in previous earthquakes in the time period from 1970 to 1994, led the structural engineers to regard it as the best and viable construction technique to resist seismic load.

However, failures in 1994 Northridge earthquake and 1995 Kobe earthquake raised major concerns on the performance of welded steel moment frames. A wide range of brittle connection failures from minor cracking to complete fracture of structural members were observed. Damage was most commonly observed near the welded joint connecting a girder bottom flange to the supporting column flange (Fig. 2.1). A partnership of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE) formed the "SAC" steel program with a goal to study welded moment connections after Northridge earthquake. Funding from Federal Emergency Management Agency (FEMA) initiated the 6-year SAC steel program to conduct research, develop solutions and investigate the damages occurred during these seismic events. Results of this research program suggested various causes of potential brittle failures including poor workmanship and inspection, poor welding procedure and filler material, over-strength of beam, stress and strain concentration near the welded joint connecting a girder bottom flange to supporting column

flange (Yang and Popov 1995 and, Kaufmann et al. 1997). Since then, additional studies and proposals have been made to rectify these failures.

2.1.1 Seismic design of moment resisting frames and braced frames

During the 1994 Northridge earthquake, steel structures that were thought to be ductile experienced brittle failures. After this event, through research, American Institute of Steel Construction (AISC) established seismic provisions that lead to the design philosophy of capacity design of steel structures. There are provisions in National Earthquake Hazards Reduction Program (NEHRP) for capacity design where the structural members are designed in such a way that they behave as fuses that dissipate energy through inelastic deformation during major earthquakes.

Due to plastic deformation, a structure can be designed for lower seismic forces compared to that deform elastically. Ductility and overstrength are key factors in calculating the reduced seismic forces used in the design. Uang and Bertero (1986) conducted several tests on steel and reinforced concrete frames, and concluded that the response modification factor or seismic force reduction factor (R) is a product of structural overstrength factor (R_0) and system ductility factor (R_d). On the other hand, ATC-19 (ATC 1995) suggested that R factor is a product of R_0 , R_d and R_R , where R_R represents factor for redundancy. Seismology committee of SEAOC proposed the inclusion of R_0 for structural overstrength and R_d for ductility reduction in the seismic force reduction factor R . Provisions in National Building Code of Canada (NBCC 2015) for seismic design consider ductility and overstrength factors, R_d and R_0 , in calculating the reduced seismic forces used in the design. Structural members are assumed to yield and dissipate energy. Different R_d

and R_0 values are prescribed for various seismic force resisting systems depending on the type of system and level of ductility. Figure 2.2 shows the different R_d and R_0 values for steel seismic systems of braced frame, moment frame, and steel plate shear wall at different ductility levels. The following are some of the key factors on the seismic forces and their effects that have to be considered in the design:

- Total seismic distributed force on the building must include the loads due to accidental torsion and notional loads.
- The actual deflection is the product of $R_d R_0 / I_E$ on the deflection of the building calculated using the reduced seismic forces, where, I_E is the importance factor of the building.
- The maximum inter-storey drift limits are 0.01h for post-disaster buildings, 0.02h for school buildings and 0.025h for other buildings, where h is the height of the storey.
- In capacity design, certain structural elements have to be selected and designed to yield and dissipate the energy.

The design base shear can be calculated for any given location according to its corresponding design acceleration spectrum from the expression

$$V = C_e \left(\frac{I_E}{R} \right) \quad (2.1)$$

where C_e is the normalized design pseudo-acceleration, I_E is the importance factor of the building, $R = R_d R_0$ is the response modification factor. According to NBCC 2015 clause 4.1.8.11 (6), the shear force is distributed to all the levels according to

$$V_x = V \frac{W_x h_x}{\sum_i^n W_i h_i} \quad (2.2)$$

where V_x is the static shear force at x^{th} level, W_x and W_i is the weight of the structure at level x and i , and h_x and h_i is the height of the structure at level x and i .

2.1.2 Moment resisting frames

Over the years, two main design strategies have been studied to address the failure of beam-column welds in the moment frames. The first method consisted of a procedure that involved the strengthening of the beam near the beam-column connection by providing additional bolted or welded plates and stiffeners. The second method involves deliberately weakening the beam by removal of a portion of the beam flange at a short distance from the column face. Thus, the plastic hinge is moved away from the beam-column connection and formed in the beam (Chi and Uang 2002).

Different types of moment connections have been discussed in Moment Connections for Seismic Applications (CISC 2014). Detailed design guidelines also have been provided for various connections such as reduced beam section (RBS) moment connection, bolted unstiffened (BU EP), stiffened (BSEP) end-plate moment connections, and bolted flange plate (BFP) moment connection. Requirements and limitations of these moment connections for use in ductile moment-resisting frame (type D), moderately ductile moment-resisting frame (type MD), and limited-ductility moment-resisting frame (type LD) are discussed. Both BU EP and BSEP connections are formed of end-plate, which is welded to the beam and bolted to the flange of the column. The primary purpose of such connections is to allow inelastic deformation of the beam by providing ample strength to the connection. In the BFP moment connection, steel plates are welded to column flanges using complete joint penetration welds and bolted to the beam flanges by

using high-strength bolts. The beam web is connected to the column flange using a shear tab. The location of initial yielding and plastic hinge formation is intended to occur near the unwelded end of the flange plates. The other connection is the reduced beam section (RBS) connection, which will be discussed in greater details in the subsequent section.

2.2 Reduced beam section connection

Reduced beam section connection consists of detailing that deliberately removes a portion of the beam at a short distance from the beam-column connection to weaken the beam and induce the formation of the plastic hinge in the beam. This type of connection has proven to be more economical than the detailing to reinforce beam-column interface as there is a reduced demand of continuity plates, panel zone reinforcement and strong column-weak beam requirements (Jin and El-Tawil 2004). The joint research project conducted by steel producer S. A. Arbed and European Union in 1992 had validated and eventually patented reduced beam section connection (Iwankiv and Carter 1996). Studies on RBS moment connection and its design provisions are discussed further in the following sections.

2.2.1 Experimental study and behavior of beam-column connection with RBS

Post Northridge earthquake, three types of cut outs consist of constant cut, tapered cut and radius cut were considered in the research and testing of RBS connections. Engelhardt et al. (1996) performed full-scale tests under cyclic loading on a straight cut RBS with fully welded beam and column. Fracture was observed near end of the flat portion of the cut close to the column face in final cycles of the experiment and was

analyzed through moment-plastic rotation response at the plastic hinge. The plastic hinge rotation at the ultimate load was calculated as

$$\theta_p = (\phi_u - \phi_y)l_p \quad (2.3)$$

where ϕ_u and ϕ_y is the curvature of beam at ultimate load and yielding load respectively and l_p is the equivalent length of plastic hinge. The experimental specimen of Engelhardt et al. (1996) managed to sustain a 0.02 radian of plastic rotation in the beam at close to the face of column in a cyclic test when the cyclic load was applied at the free end of the cantilever beam. The cyclic test conducted by Iwankiv and Carter (1996) on a tapered cut RBS reached a maximum plastic rotation of 0.04 radian in the beam at close to the face on column in the fifth displacement cycle applied at the tip of cantilever. For this case, fracture was observed in the narrowest section of the beam flange in final cycles. Subsequent experiments conducted by Popov et al. (1998) and Engelhardt (1998) on radius cut exhibited a gradual strength degradation of the members and sustained brittle fracture. The maximum plastic rotation at the center of RBS exceeded 0.03 radian which is rare in a moment resisting frames subjected to ultimate load.

Experimental research and analytical studies were conducted by Chi and Uang (2002) to investigate the cyclic response on RBS moment connections with radius cut and deep columns. A cyclic displacement was applied at the end of the beam. Deep columns were employed in a moment resisting frame mainly to limit the overall drift. A plastic rotation of 0.03 radian in the beam at close to the face of column has been achieved by the tests. However, out-of-plane bending and twisting of columns were observed in all tests. These were mainly due to the lateral buckling of the beam with RBS and warping in the column. Chi and Uang (2002) suggested to provide sufficient lateral bracing near the RBS

to prevent these failures.

Different beam web connections of RBS connections to the column have also been studied. Jones et al. (2002) found that welded web connections performed better compared to the bolted web connections as it reduces the vulnerability of fracture at the flange connection. Studies on bolted web specimens conducted by Lee and Kim (2004) found that premature brittle failure of the beam flange occurred at the weld access hole. Lee and Kim (2004) recommended a new design procedure for beam web bolted connections for RBS connections to overcome this deficiency. The procedure suggested to prevent the bolt slippage by designing the bolt group against eccentric loading. Hence, a higher slip-critical bolt requirement than the earlier design method was recommended. The specimen designed according to the new procedure exhibited a plastic rotation capacity of 0.05 radian without fracture at the beam-column connection.

2.2.2 Design considerations for RBS moment connection

The size and shape of RBS are critical in controlling the moment developed in the system and at the face of the column. Figure 2.3 shows the typical geometry of radius cut RBS where, a is the transition length representing the distance between the face of the column to the RBS, b is the total length of RBS and c is the depth of RBS cut. There was no standard protocol on how to determine these dimensions. However, Engelhardt et al. (1998) stressed the importance of these dimensions on the performance of RBS. The transition length a should be large enough so that the stress is uniform across the flange width at the face of the column. The length of RBS b should be large enough to avoid excessive inelastic strains in the RBS and the cut depth c should be chosen such that the

maximum moment formed at the column face is between 85% to 100% of the plastic moment of the beam. It was suggested that the depth of the flange cut $2c$ should be no greater than 50% of the beam flange. Moment Connections for Seismic Application by CISC (2014) recommends the reduction to be between 20% to 50% of the beam flange.

2.2.3 Numerical studies of beam-column connection with RBS

Along with experimental studies, some numerical simulations have been carried out to study the RBS connection. Zhang and Ricles (2006) carried out finite element analysis that replicated the experimental study. As shown in Fig. 2.4, the test setup consisted of beams on the right and left side of the column with RBS at a distance of 229 mm from the face of column on each side of the beam. The bottom end of the column is pin supported. The far end of each beam from the column is supported by a rigid link that acts as a roller support. The beam section size was selected such that the assembly followed the strong column weak beam protocol according to American Institute of Steel Construction (AISC 2016) seismic provisions. A finite element modelling was carried out in ABAQUS computer program (ABAQUS 2017) using elastic-plastic compression-only material model. Results of the finite element analyses were found to be in good agreement with the experimental results.

Li et al. (2009) carried out an analytical study to find solutions to prevent early strength deterioration due to local buckling in the RBS region of the beam. A bilinear material model with kinematic strain hardening having the second stiffness of $E/100$ and yield strength of 235 MPa was used. The analysis was repeated with yield strength of 345 MPa. It was found that providing stiffeners at the RBS can delay the local buckling of the

beam. The study suggested to provide two pair of stiffeners at the RBS portion to achieve plastic rotation up to 0.03 radian and three pairs for up to 0.045 radian of plastic rotation at the beam flange near the face of the column. However, these numerical studies were not validated with experimental tests.

2.2.4 Study of building with RBS

Shen et al. (1999) investigated the performance of special moment resisting frames of pre-Northridge type with RBS by conducting non-linear static and time-history analyses on three and ten storey MRFs with varying RBS configurations. The analyses were carried out with the software DRAIN-2DX (Prakash et al. 1993) with shear failure criteria. The shear strength, elastic stiffness and strain hardening stiffness for the material model are obtained from the equations proposed by Krawinkler (1978). Some parameters of the equations are

$$V_y = 0.55F_y d_c t \quad (2.4)$$

$$\gamma_y = \frac{F_y}{\sqrt{3}G} \quad (2.5)$$

$$V_u = 0.55F_y d_c t \left(1 + \frac{3b_c t_{cf}^2}{d_b d_c t} \right) \quad (2.6)$$

where F_y is the tensile yield strength, G is the shear modulus, d_c , b_c , t and t_{cf} are the depth, flange width, web and flange thickness of the column, d_b is the depth of the beam, V_u is the shear strength, V_y is the shear yield strength and γ_y is yield distortion.

This study focused on improving the efficiency of RBS by choosing the optimal reduction ratio of beam flange and the location of RBS. Results from the dynamic analyses suggested negligible increase in inter-storey drift when the RBS flange reduction was up

to 40% of the flange width. However, when flange reduction reached 60% of the flange width, 15% to 20% increase in the inter-storey drift and 20% reduction in strength of the frames were observed. In this study, it was also concluded that a balanced performance of RBS is obtained when the transition length of the RBS is between 15% to 75% of the depth of the beam and RBS flange reduction is between 40% to 60%.

Jin and El-Tawil (2004) have carried out analytical studies on the non-linear pushover and transient behavior of moment frames with various storey heights. The research conducted by Jin and El-Tawil (2004) consisted of non-linear pushover and transient analyses of 4, 8 and 16 story MRFs with RBS modelled in software DYNAMIX (El-Tawil and Deierlein 2001) with kinematic strain hardening material properties. Inter-storey drift angle results obtained from the pushover analyses suggested that frames with RBS connections provide acceptable performance in the areas of high seismic risk.

2.3 Material response

In a severe earthquake event, a steel structure can experience significant yielding and a high rate of deformation. Thus, it is important that deformation (strain) rate dependency of the material properties and plastic behavior of the material are properly considered and modelled in assessing the response of the structure.

Strain rate is the change in strain or deformation with respect to time defined as

$$\dot{\varepsilon} = \frac{1}{L} \frac{dL}{dt} = \frac{d\varepsilon}{dt} \quad (2.7)$$

where ε is the strain, L is the length and t is the time. Tests by Manjoine (1944) on structural steel showed that yield strength increases with the increase in strain rate. It was observed that the ratio of increase with the strain rate was found to be lower at the ultimate strength

than at initial yielding (yield strength). However, the elastic modulus of steel has been found not to be affected by strain rate (Wakabayashi et al. 1984 and Chen 2010).

2.3.1 Plasticity

Many plasticity constitutive models have been proposed to model metals including steel. One of the models is the combined isotropic and kinematic hardening model by Armstrong and Frederick (1966) and modified by Chaboche et al. (1979). Studies by Krempl (1979), and Chang and Lee (1987) have shown that this model is able to closely represent the stress-strain or load-deformation response of steel subjected to both monotonic and cyclic loadings. According to Lemaitre and Chaboche (1990), the yield surface can be defined for the von Mises yield criterion as

$$f(\sigma_{ij} - \alpha_{ij}) = \sqrt{\frac{3}{2} (s_{ij} - \alpha_{ij})(s_{ij} - \alpha_{ij})} - K_0 = 0 \quad (2.8)$$

where σ_{ij} is the stress tensor, α_{ij} is the back stress representing the center associated with the translation of yield surface, s_{ij} is the deviatoric stress and K_0 is the variable that defines the size of yield surface. The plastic strain tensor evolves according to

$$de_{kl}^p = \frac{\partial f(\sigma_{ij} - \alpha_{ij})}{\partial \sigma_{kl}} d\lambda \quad (2.9)$$

where $d\lambda$ is a constant of proportionality. The back stress tensor consists of the summations of N number of back stress tensor terms given by

$$\alpha_{ij} = \sum_{n=1}^N \alpha_{ijn} \quad (2.10)$$

where the n^{th} back stress tensor varies according to

$$d\alpha_{ijn} = c_n \frac{1}{K_0} (\sigma_{ij} - \alpha_{ij}) d\varepsilon^{pl} - \gamma_n \alpha_{ijn} d\varepsilon^{pl} \quad (2.11)$$

where C_n and γ_n are constants that characterize the evolution of the n^{th} term back stress tensor and the change in the equivalent plastic strain given by

$$d\varepsilon^{pl} = \sqrt{\frac{2}{3}} d\varepsilon_{kl}^p d\varepsilon_{kl}^p \quad (2.12)$$

Under uniaxial monotonic tension loading with zero initial back stress, Eq. (2.11) becomes

$$d\alpha_n = C_n d\varepsilon_t^p - \gamma_n \alpha_n d\varepsilon_t^p \quad (2.13)$$

where α_n is the n^{th} term equivalent uniaxial back stress given by

$$\alpha_n = \sqrt{\frac{3}{2}} \alpha_{kln} \alpha_{kln} \quad (2.14)$$

and ε_t^p is the uniaxial true plastic strain calculated from Eq. (2.9) under monotonic uniaxial tension loading. Under uniaxial monotonic tension loading, Eq. (2.10) can be integrated to give

$$\alpha_n = \frac{C_n}{\gamma_n} \left(1 - e^{-\gamma_n \varepsilon_t^p} \right) \quad (2.15)$$

The size of yield surface can be defined using the equivalent isotropic yield stress as

$$\sigma_{eq}^i = \sqrt{\frac{3}{2}} (s_{kl} - \alpha_{kl})(s_{kl} - \alpha_{kl}) = K_0 \quad (2.16)$$

Under uniaxial tension loading, the flow stress is a combination of isotropic yield stress and back stress given by

$$\sigma_t^F = \sigma^i + \alpha \quad (2.17)$$

where σ_t^F is the flow stress that is a function of true plastic strain ε_t^p , σ^i is the isotropic yield stress that is equal to σ_{eq}^i for uniaxial condition, and α is the back stress given by

$$\alpha = \sum_{n=1}^N \alpha_n \quad (2.18)$$

Tests and studies by Krempl (1979), Chang and Lee (1987), Lemaitre and Chaboche (1990), and Walker (2012) found that the flow stress is affected only by the current strain rate, and not the strain rate history. Thus, kinematic hardening can be considered not to be affected by strain rate.

2.3.2 Modelling of strain rate effect

Many researchers have proposed equations to take into account effects of strain rate in modelling the material strength. Some of these are discussed in the following section.

Cowper and Symonds model (1957) proposed a power law equation based on the experimental data for copper and steel that relates the yield stress at different strain rates with static yield stress as

$$\sigma_y(\dot{\epsilon}) = \sigma_y \left(1 + \left(\frac{\dot{\epsilon}}{D} \right)^k \right) \quad (2.19)$$

where D and k are material constants, $\sigma_y(\dot{\epsilon})$ is the yield stress at $\dot{\epsilon}$ strain rate and σ_y is the static yield stress.

Wang et al. (1999) conducted tests on metal alloy. The study suggested a linear relationship between the flow stress and logarithmic strain rate as

$$\sigma_y(\dot{\epsilon}) = A + B \ln \dot{\epsilon} \quad (2.20)$$

where $\sigma_y(\dot{\epsilon})$ is the yield strength, $\dot{\epsilon}$ is the strain rate and, A and B are material constants. However, this equation is valid only for up to intermediate strain rate.

Othman (2015) proposed an improvement to the proposed equation by Eyring (1936) equation of

$$\sigma_y(\dot{\epsilon}) = \sigma_0 + \frac{k_b T}{V_0} \ln \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right) \quad (2.21)$$

where $\sigma_y(\dot{\epsilon})$ is the yield stress at a strain rate $\dot{\epsilon}$, σ_0 is the yield stress at the reference strain rate $\dot{\epsilon}_0$, V_0 is a constant, and k_b is the Boltzmann constant. In the modified equation, an exponential term was added to Eq. (2.21) as

$$\sigma_y(\dot{\epsilon}) = \sigma_0 + \frac{k_b T}{V_0} \left(\exp \left(\sqrt{\frac{\dot{\epsilon}}{\dot{\epsilon}_c}} \right) \ln \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right) \right) \quad (2.22)$$

where $\dot{\epsilon}_c$ is the critical strain rate.

Salahi and Othman (2016) conducted a comparative study of a number of proposed equations that relate the yield stress to the strain rate. It was suggested that either Eq. (2.19) by Cowper and Symonds (1957) or Eq. (2.21) by Othman (2015) be used to model the effects of strain rate on yield stress as these equations require only a few material constants and fitted that test data well for a large range of strain rate.

2.3.3 Strain rate effect on steel under earthquake loading

There have been various studies conducted to understand the behavior of steel under earthquake loading. Chang and Lee (1987) stated that the strain rate in steel structures can reach up to 10^{-1} s^{-1} under severe seismic activities. Wakabayashi et al. (1984) investigated the level and effect of strain rate in structures during an earthquake. Dynamic and quasi-static tests of the steel specimens were tested at different strain rates to understand the uniaxial stress-strain relationship under cyclic axial loading. From the experimental results, the yield stress (σ_y) was assumed to increase proportionally with the logarithmic strain rate, which can be expressed as

$$\frac{\sigma_y}{\sigma_{y0}} = 1 + 4.73 \times 10^{-2} \log_{10} \left| \frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right| \quad (2.23)$$

where σ_{yo} is the quasi-static yield stress at the rate of $\dot{\epsilon}_0 = 5 \times 10^{-5} \text{ s}^{-1}$. The results suggested that the maximum strain rate in the structure increases with the increase in the flexibility of the structure.

Zhang and Li (2011) studied the dynamic behavior of reinforced concrete structures subjected to earthquake loading. The yield strength of steel bars was assumed to vary with strain rate as

$$f_{yd} = \left(1 + c_f \lg \frac{\dot{\epsilon}}{\dot{\epsilon}_0}\right) f_{ys} \quad (2.24)$$

where f_{yd} is the dynamic yield strength, $\dot{\epsilon}$ is the current strain rate, $\dot{\epsilon}_0$ is the quasi-static strain rate and c_f is a parameter represented by the following expression,

$$c_f = 0.1709 - 3.289 \times 10^{-4} f_{ys} \quad (2.25)$$

where f_{ys} is the yield strength at quasi-static strain rate. It was found that the maximum base shear and base moment increase when strain rate dependent material properties were included in the analyses.

Li and Li (2011) studied the effects of strain rate on reinforced concrete structure under earthquake loading. Various grades of reinforcing steel were tested at various strain rates ($2.5 \times 10^{-4} \text{ s}^{-1}$ to 0.1 s^{-1}) to study the strain rate effects. The effect of strain rate on the reinforcing steel was modelled with Eq. (2.24). The stress versus strain curves at various strain rates for monotonic loading tests suggested an increase in yield strength, ultimate strength, strain at the beginning of strain hardening and length of yield plateau increase with the increase in strain rate. A reinforced concrete structure was also analysed under three earthquake records considering the strain rate effects on reinforcing steel. When results of analyses with the consideration of strain rate effects were compared to that without the consideration of strain rate effects, the maximum compressive axial force,

maximum base shear and maximum bending moment at the base node were observed to increase. However, the top displacement was observed to decrease when effects of strain rate were considered.

Tian et al. (2014) studied the effects of strain rate on the seismic response of the transmission tower. The tower was calculated by using incremental dynamic analysis method with and without consideration of strain-rate dependent material properties. The elastic-perfectly plastic material model as shown in Fig. 2.5 was used in analyses. The yield strength was assumed to vary with the strain rate according to Eq. (2.24). Three earthquake records were scaled up until a progressive collapse of the structure occurs. The results of the analyses show that the maximum base shear increases when strain rate effects were included in the analyses.

Singh (2019) investigated effects of strain rate on the forces and moments generated in a moment frame with reduced beam section due to earthquake loading. Material properties used in the dynamic analyses were calibrated from tests by Chen (2010) and Walker (2012). The analyses were carried out using mixed mode hardening model by Chaboche et al. (1979) with the dynamic amplification factor on the isotropic yield stress given by

$$D_a = \frac{\sigma^i(\dot{\epsilon})}{\sigma_s^i} = 1 + S(\dot{\epsilon})\xi_{pl} \quad (2.26)$$

where $\sigma^i(\dot{\epsilon})$ is the isotropic yield stress at the strain rate $\dot{\epsilon}$ and σ_s^i is the static isotropic yield stress. $S(\dot{\epsilon})$ is the term from Cowper and Symonds (1957) from Eq. (2.19) rearranged as

$$S(\dot{\epsilon}) = \left(\frac{\dot{\epsilon}}{I} \right)^J \quad (2.27)$$

where I and J are constants, and ξ_{pl} is a term added to Eq. (2.19) to model the reduction in stress amplification with plastic strain according to

$$\xi_{pl} = 1 + Ae^{-B(\varepsilon_t^p - \varepsilon_0^p)} \quad (2.28)$$

where A and B are constants, ε_t^p is the true plastic strain and ε_0^p is the true plastic strain at the end of yield plateau. Results of the dynamic analyses showed that moments and forces generated can increase by up to 10% when strain rate effects on the material properties were considered in the analyses.

2.4 Concluding remarks

There are few studies conducted to analyze the performance of a moment resisting frames (MRF) with reduced beam section (RBS). Most of the studies either have not considered effects of strain rate or have not utilized material properties that are representative of the actual material that can provide a more accurate prediction of the response of MRFs with RBS subjected to seismic loading. Furthermore, analyses with strain rate effects have not considered the influence of design ductility. Thus, there is a need to investigate the influence of design ductility on the performance of MRF with reduced beam section using more representative material properties that include effects of strain rate.

Figure 2.1: Common fracture zone in a typical beam-column connection (Malley et al. 2004)

Figure 2.2: Flowchart of steel seismic system (Metten and Driver 2015)

Figure 2.3 Geometry of a typical radius cut RBS connection (AISC 2016)

Figure 2.4: Experimental test setup for evaluation of RBS connection to deep column under cyclic loading (Zhang and Ricles 2006)

Figure 2.5 : Stress versus strain curve representing elastic-perfectly plastic material model (Tian et al. 2014)

Chapter 3: Numerical modelling of the moment resisting frame

The performance and behavior of moment resisting frames (MRF) with reduced beam section (RBS) are being investigated in this study for effects due to strain rate dependency of the material properties. There have been studies that look into the behavior of moment resisting frames with RBS. Among them, Jin and El-Tawil (2004) have designed and analysed 4, 8 and 16 stories moment resisting frames with RBS subjected to a suite of recorded earthquake ground motions. In this study, four storey high moderately ductile and limited-ductility moment resisting frames are being considered. These frames are designed based closely on building dimensions of Jin and El-Tawil (2004). The design of these frames and the corresponding finite element models developed for this study are presented.

3.1 Material properties

It is critical that realistic material properties be used when performing the numerical simulations in order to obtain results that are more representative of the actual structure. Singh (2019) has calibrated material properties parameters for the mixed-mode hardening model by Lemaitre and Chaboche (1990) from monotonic and cyclic steel coupon tests for two grades of steel by Chen (2010) and Walker (2012). The two grades of steel were CAN/CSA G40.20/21 300W and ASTM A572 grade 50, which are designated as material G and material H respectively. For this study, numerical simulations have been carried out using materials G and H. Material properties and parameters of these materials for the hardening model calibrated by Singh (2019) that are used in the analyses are presented. The elastic modulus and Poisson's ratio are taken 200000 MPa and 0.3 for all the analyses.

According Singh (2019) the isotropic yield stress versus plastic strain curve under

uniaxial monotonic tension can be taken as

$$\sigma_s^i = \sigma_s^{yp} + \sigma_1 [1 - \exp(-a (\varepsilon_t^p - \varepsilon_t^o)^b)] \text{ for } (\varepsilon_t^p - \varepsilon_t^o) > 0 \quad (3.1)$$

where σ_s^{yp} is the isotropic yield stress at the end of yield plateau, σ_s^i is the isotropic yield stress, σ_1 is additional increase in the yield stress over σ_s^{yp} at the infinite finite plastic strain such that $\sigma_s^{yp} + \sigma_1$ is equal to saturated yield stress, ε_t^p is the true plastic strain, ε_t^o is plastic strain at the end of yield plateau, a and b are constants. The amplification of isotropic yield stress is given by

$$\sigma_t^F(\dot{\varepsilon}) = [(1 + S(\dot{\varepsilon})\xi_{pl})\sigma_s^i] = (D_a \sigma_s^i) \quad \text{with} \quad (3.2)$$

$$S(\dot{\varepsilon}) = \left(\frac{\dot{\varepsilon}}{I}\right)^{\frac{1}{J}} \quad \text{and} \quad (3.3)$$

$$\xi_{pl} = (1 + Ae^{-B(\varepsilon_t^p - \varepsilon_t^o)}) \quad (3.4)$$

where D_a is the isotropic yield stress amplification factor and, I, J, A and B are constants. $S(\dot{\varepsilon})$ defines the isotropic yield stress amplification factor at a large plastic strain (infinite) and ξ_{pl} gives the reduction in the amplification factor with plastic strain. Four kinematic hardening terms are used to model the material to give the total back stress as

$$\alpha = \sum_{n=1}^4 \frac{C_n}{\gamma_n} (1 - e^{-\gamma_n \varepsilon_t^p}) \quad (3.5)$$

where C_n and γ_n are constants for back stress term n . The flow stress at a constant strain rate is given by

$$\sigma_t^F(\dot{\varepsilon}) = \sigma^i(\dot{\varepsilon}) + \alpha \quad (3.6)$$

Parameters used in generating the material properties are shown in Table 3.1. Materials G and H have the initial static yield stress of 343 MPa and 373 MPa, which are

closer to the nominal yield stress of grade 350W steel and the corresponding probable yield stress.

Figures 3.1 to 3.5 show the isotropic yield stress, flow stress amplification factor, isotropic yield stress, back stress and, flow stress versus plastic strain at a few strain rates for materials H and G. It can be seen that material G has a higher stress amplification due to strain rate but an overall lower strength. For material G, the initial isotropic yield stress amplification is around 1.4 and flow stress amplification is around 1.23. The amplification factor decreases with plastic strain.

3.2 Moment resisting frame

Dimensions of the four-storey building considered by Jin and El-Tawil (2004) are shown in Figs. 3.6 and 3.7. Locations of MRFs and dimensions of the building used in this study are shown in Fig. 3.8. There are two moment resisting frames on each side of the perimeter with the moment and non-moment frames arranged alternatively, and MRFs on each side of the building are identified with “a”. The span of the bay is slightly narrower at 9 m instead of 9.14 m and the storey heights are all 4 m instead of 3.66 m and 4.57 m. The building is assumed to be located at Powell River, British Columbia, on a class D site, which is in a high seismic hazard zone.

In the study by Singh (2019), ductile MRF considered was designed only for strength, and not the inter-storey drift limit. Thus, the inter-storey drift of the frame was much larger than the 2.5% limit. It was found that the column has to be significantly oversized for strength in the ductile MRF in order to reduce the inter-storey drift to below the limit. Thus in this study, two moderately ductile and one limited ductility MRFs are

considered. One moderately ductile MRF is designed with oversized columns for strength to meet the inter-storey drift limit and another one just for strength alone. These frames are respectively identified as MD1 and MD2. MD2 can apply to the applications where non-structural components will not likely experience significant damage with a large inter-storey drift such as pipe racks. The single limited ductility MRF considered is designed for strength and to meet inter-storey drift limit, and is identified as LD1.

Before designing the four-storey frame, a single storey moderately ductile moment resisting frame is first being used to carry out preliminary studies to decide on the size of RBS and finite element model to be considered for the four-storey building in this study. The validation of the material properties and the type of element used in the modelling of the frames are carried out in Chapter 4. It has the same storey height, and bay span as the four storey building, and with the same placement of the moment resisting frames. Both the single storey and four storey frames are designed according to Moment Connection for Seismic Application (CISC 2014), National Building Code of Canada (NBCC 2015) and CSA S16-14 Design of Steel Structures (CSA 2014). Lateral seismic forces acting on each storey are calculated using the equivalent static method for seismic design. The loading and design of four-storey moment resisting frames are provided in Appendix A, and that for single storey frames in Appendix B. The four storey frames have been designed with the maximum flange cut ratio of RBS close to 50%.

3.2.1 Preliminary study - single storey moderately ductile moment resisting frame

Two single storey moderately ductile moment resisting frames with reduced beam section (RBS) having two different maximum flange width cut ratios are considered.

Detailed design of these frames is provided in Appendix B. Frame PM1 has a maximum reduction of 46% in the flange width, and frame PM2 has a 32% maximum reduction. Frame PM1 is also being used to conduct mesh sensitivity analyses to determine the optimum mesh configuration. Dimensions of the frames and RBS are shown in Figs. 3.9 and 3.10. The beam and columns are modelled with Timoshenko beam element B31OS which is a two node linear open section beam, and the columns are hinged supported at the base. A leaning column is provided to capture the P-delta effect due to the gravity load. A stiff rectangular cross-section with Timoshenko beam element B31, a two node linear beam, is used for the leaning column. The lateral load of the leaning column is transferred to the frame with the command *EQUATION. Material properties of material H is provided in Section 3.1 are used in the analyses.

A classical Rayleigh damping matrix proportional to both stiffness and mass is considered in the analyses, which is

$$[C] = \alpha[M] + \beta[K] \quad (3.7)$$

where $[C]$ is the damping matrix, $[M]$ is the mass matrix, $[K]$ is the stiffness matrix, α and β represent coefficients which correspond to mass and stiffness matrices respectively. The damping coefficients can be calculated for a generalized mass proportional damping and stiffness proportional damping system for n^{th} mode

$$C_n = \alpha M_n + \beta K_n \quad (3.8)$$

where M_n and K_n are the generalized mass and stiffness of the system for the n^{th} mode respectively. The modal damping ratio (ε) for the n^{th} mode can be determined from

$$\varepsilon_n = \frac{C_n}{2M_n\omega_n} \quad (3.9)$$

where $\omega_n = \sqrt{K_n/M_n}$ is the natural frequency of the system for the n^{th} mode.

From Eq. (3.8) with $\beta = 0$, the modal damping ratio (ε) for the mass proportional damping system can be expressed as

$$\varepsilon_n = \frac{\alpha}{2\omega_n} \quad (3.10)$$

Similarly from Eq. (3.8) for $\alpha = 0$, the modal damping ratio (ε) for the stiffness proportional damping system can be expressed as

$$\varepsilon_n = \frac{\beta\omega_n}{2} \quad (3.11)$$

Combining Eqs. (3.10) and (3.11), the damping ratio for Rayleigh damping as expressed in Eq. (3.5) becomes

$$\varepsilon_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \quad (3.12)$$

The coefficients α and β can be determined for the specified damping ratios ε_i and ε_j for the i^{th} and j^{th} modes can be expressed in a matrix form as

$$\begin{Bmatrix} \varepsilon_i \\ \varepsilon_j \end{Bmatrix} = \frac{1}{2} \begin{bmatrix} 1/\omega_i & \omega_i \\ 1/\omega_j & \omega_j \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} \quad (3.13)$$

The coefficients of damping can be obtained by solving the two algebraic equations. With the damping ratio (ε) assumed to be the same for both the modes, Song and Su (2017) gave α and β as

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\varepsilon}{\omega_i + \omega_j} \begin{bmatrix} \omega_i\omega_j \\ 1 \end{bmatrix} \quad (3.14)$$

Orban (2011) and Kudu et al. (2015) suggested that damping ratio for the majority of the steel structures is around 3%. Hence, 3% of Rayleigh damping ratio is used in the Eq. (3.14) to calculate the values of α and β . These values are applied in the analyses through the

material properties input.

The fundamental time period of PM1 is 1.42s and frequencies for the first two modes are $\omega_1 = 4.42$ rad/s and $\omega_2 = 98.12$ rad/s. Values of damping constants α and β calculated for this model are 0.25 rad/s and 5.85×10^{-4} s/rad respectively. For PM2, the time period is 1.08s. Values of damping constants calculated from the first two modes of $\omega_1 = 5.81$ rad/s and $\omega_2 = 130.83$ rad/s are $\alpha = 0.33$ rad/s and $\beta = 4.39 \times 10^{-4}$ s/rad. These models have been analyzed under arbitrarily chosen seismic ground motion RSN 15 from the suite of 11 earthquake records and scaling procedure outlines in Chapter 5. Details for this earthquake record is given in Chapter 5. The total scaling factor on the ground motion for PM1 is 2.56, and PM2 is 2.68.

3.2.2 Mesh convergence study for preliminary model

The accuracy of the results and time taken to run the analysis depend on the type of element used in the modelling. While shell element only model and hybrid element model consists of beam and shell (at RBS) elements can give a better accuracy when compared to the beam element only model, they also require significantly much longer time to run the analyses. Singh (2019) found that hybrid element model took more than ten times longer than the beam element only model to run the analysis, but there were only small differences in the results. Thus following Singh (2019), beam element only models are used for the analyses in the current study. Several mesh schemes have been investigated for different parts of the model such as the RBS region, the transition zone between RBS and the column, the rest of the beam and columns in order to find the optimum mesh configuration.

A plastic hinge is formed at the RBS when the MRF is subjected to an intense

seismic ground motion. Due to the high rate of change in the cross-section, and as well as the stress and strain, it is necessary to have a fine mesh to capture these effects at the region of radius cut of RBS. As shown in Fig. 3.11, the RBS profile can be approximated by dividing the cut region into segments with varying flange width to form the radius cut shape. Singh (2019) gave the expression to calculate the average width of the beam element segment as

$$b_i = \frac{1}{z_{i+1} - z_i} \int_{z_i}^{z_{i+1}} b(z) dz \quad (3.15)$$

where $b(z)$ is the width of the flange along the longitudinal direction, and z_i and z_{i+1} are the corresponding coordinates at the start and end of the i^{th} beam element. The base mesh scheme considered for each column for mesh sensitivity analysis of RBS (cut) region consists of 10 elements at RBS region and 2 elements at the transition zone as noted in Fig. 3.12 for various region, and 80 elements for the rest of the beam and 2 elements for each column. Material properties of material H are used in the mesh sensitivity analyses. Three mesh schemes with 20 elements, 40 elements and 80 elements at the RBS region have been considered. A number of parameters from results of the analyses for these three schemes are shown in Table 3.2. The difference in the result between the three schemes are small. The largest difference in the results between the 20 and 40 element scheme is 0.9% for the inter-storey drift and displacement. The difference between results of 40 and 80 element schemes are even smaller, at less than 0.5%. Therefore, mesh scheme 2 with 40 elements at a width of 7.5 mm in the region of the RBS cut has been selected to model the RBS cut region in the study.

For the transition zone “a” from the face of the column to the start of RBS cut, four schemes of 2, 5, 10 and 20 elements are considered. Results of the analyses for a number

of parameters are shown in Table 3.3. There is hardly any difference in the results for all four schemes. Although the scheme with 2 elements appear to be sufficient, the element size appears to be coarse. Thus, scheme 2 with 5 elements has been selected to provide a smaller element size in order to avoid any problem when modelling the four storey frame.

As shown in Table 3.4, four schemes of various element numbers are considered for the rest of the beam, which includes the segment between the two RBS and the segment between transition zone and the center of the column. It can be seen in Table 3.5 that there is very little difference between the results of the analyses for the four schemes. All four schemes appears to be acceptable. Nevertheless, scheme 3 in Table 3.4 with the element size that closely matches the element size for the transition zone has been selected. Scheme 3 has 10 elements between the column centerline and the transition zone, and 320 elements between the two RBS. There should also be a transition zone between the RBS cut region towards the center of the beam. This is not considered explicitly since the element sizes chosen for the rest of the beam and transition zone towards to the column are close.

Finally, a total of 5 schemes with 2, 25, 50, 100 and 200 elements have been considered for the column. Results of the analyses are shown in Table 3.6. As can be seen in Table 3.6, there are only small differences between results of the 5 schemes. There is slightly over 1% difference in the maximum roof displacement between schemes 1 and 2, and schemes 2 and 3. The difference is even smaller between schemes 3 and 4, and hardly any noticeable difference between schemes 4 and 5. Thus instead to choosing scheme 3 that has element size about four times that of the beam, scheme 4 with 100 elements at the element size about only twice that of the beam has been selected in order to provide a closer match to the beam element size. The final scheme chosen for the single storey MRF is

shown in Fig. 3.12 for the RBS and Fig. 3.13 for the entire MRF.

3.2.3 Effects of RBS flange width reduction

According to Moment Connection for Seismic Application (CISC 2014), the recommended maximum cut of the RBS ranges from 20% to 50% of the flange width. Two single storey moment resisting frames with RBS, PM1 and PM2, having the respective RBS maximum flange cut ratios of 46% and 32% have been considered in the analyses to assess the influence of the cut ratio on the effect of material properties strain rate dependency. The design of frames PM1 and PM2 has been provided in Appendix B.

Analyses have been carried out with strain rate and non-strain rate dependent (static) material properties of materials H and G. It is expected that the frame with a larger maximum flange cut ratio of the RBS to show greater effects of material properties strain rate dependency since the RBS will experience a higher strain concentration and strain rate. Results of the maximum moment at the center of RBS for frames PM1 and PM2 are shown in Table 3.7. PM1 and PM2 have probable moments (M_{pr}) of 188 kN.m and 287 kN.m respectively. It can be seen that the rate of increase in the moment at the center of RBS due to strain rate dependency of the material properties is higher for the PM1, which has a higher maximum flange cut ratio of the RBS. The rate of increase is close to 1% higher for material G that has a higher strain rate amplification factor on the isotropic yield stress. Thus, all the four-storey frames have been designed with the maximum flange cut ratio of the RBS close to 50% in order to obtain maximum effects of material properties strain rate dependency from the analyses.

3.3 Assessment of mesh scheme for four-storey frame

The element (mesh) scheme selected for single storey moderately ductile MRF in Section 3.2.1 has been applied to four-storey moderately ductile MRF MD1, and shown in Fig. 3.14. Detailed design of MD1 has been provided in Appendix A. In the scheme selected in Section 3.2.1, all elements used are much smaller than required except at the RBS cut. It is expected that changes in the results due to the mesh refinement in the first floor RBS can be used to assess the acceptability of the mesh scheme since the first floor beam (RBS) has the biggest moment and largest in size. Thus, two schemes with 40 elements and 80 elements at the region of RBS cut for the beam at first floor have been considered.

Analyses have been carried out with material H according to the material properties and parameters provided in Section 3.1, and subjected to the seismic ground motion RSN 15 similar to the single storey frame that has a total scaling factor of 2.39. Results of the analyses of the two schemes for various parameters are shown in Table 3.8. It can be seen that there is very little difference between results of both schemes, and with the maximum difference of only 0.41% for the maximum inter-storey drift of the third floor. Thus, the element (mesh) scheme shown in Fig. 3.14 with 40 elements at the RBS cut region has been adopted to be used in the analyses of the four-storey MRF in this study.

Table 3.1: Parameters used in generating material properties

	Material G	Material H
σ_s^{yp}	194.1	249.4
σ_1	125.9	125.6
a	5.40	4.50
b	0.54	0.53
ε_t^o	0.013	0.006
I	4684	10573
J	5.04	5.49
A	1.13	0.39
B	13.1	8.0
C_1	19852	27427
γ_1	175.2	199.3
C_2	1874	692
γ_2	6.71	5.35
C_3	222.7	42976
γ_3	9.73	2466
C_4	24610	1016
γ_4	1222	9.26

Table 3.2: Comparisons of results for different mesh schemes at RBS cut (s)

Mesh scheme	1	2	3
Number of elements	20	40	80
Size of element (mm)	15	7.5	3.75
Maximum moment at the center of RBS (kN.m)	205	205	205
Maximum moment at face of the column (kN.m)	222	223	223
Maximum top displacement (mm)	64.8	65.3	65.4
Maximum inter-story drift (%)	2.19	2.21	2.21
Maximum base shear (kN)	56.71	56.80	56.85

Table 3.3: Comparisons of results for different mesh schemes at the transition zone (a)

Mesh scheme	1	2	3	4
Number of elements	2	5	10	20
Size of element (mm)	47.50	23.75	11.88	5.94
Maximum moment at the center of RBS (kN.m)	205	205	205	205
Maximum moment at face of the column (kN.m)	223	223	223	223
Maximum top displacement (mm)	65.3	65.3	65.3	65.3
Maximum inter-story drift (%)	2.21	2.21	2.21	2.21
Maximum base shear (kN)	56.80	56.80	56.80	56.80

Table 3.4: Element sizes for various mesh schemes for beam

Mesh scheme	Column centerline to column face		Rest of beam – RBS to RBS	
	Number of elements	Element size (mm)	Number of elements	Element size (mm)
1	2	99.75	80	97.64
2	5	39.90	160	48.8
3	10	19.95	320	24.41
4	20	9.98	640	12.20

Table 3.5: Comparisons of results for different mesh schemes for the rest of the beam

Mesh scheme	1	2	3	4
Maximum moment at the center of RBS (kN.m)	205	205	205	205
Maximum moment at face of the column (kN.m)	223	223	223	222
Maximum roof displacement (mm)	65.6	65.6	65.6	65.6
Maximum inter-story drift (%)	2.21	2.21	2.21	2.20
Maximum base shear (kN)	56.81	56.81	56.80	56.80

Table 3.6: Comparisons of results for different mesh schemes for sensitivity of column

Mesh scheme	1	2	3	4	5
Number of elements	2	25	50	100	200
Size of element (mm)	2250	180	90	45	22.5
Maximum moment at the center of RBS (kN.m)	203	205	205	205	205
Maximum moment at face of the column (kN.m)	220	222	223	223	223
Maximum roof displacement (mm)	67.1	66.4	65.3	65.2	65.2
Maximum inter-story drift (%)	2.26	2.25	2.21	2.21	2.21
Maximum base shear (kN)	56.10	56.60	56.80	56.80	56.80

Table 3.7: Predicted moment at the center of RBS for PM1 and PM2

Frame	Material H					Material G				
	Static, M_s (kN.m)	Rate, M_r (kN.m)	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M_s (kN.m)	Rate, M_r (kN.m)	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
PM1	197	205	4.1	1.05	1.09	191	203	6.3	1.02	1.08
PM2	299	310	3.7	1.04	1.08	290	305	5.2	1.01	1.06

Table 3.8: Comparisons of results for different mesh schemes for MD1

	Mesh-2	Mesh-3	Percentage increase (%)
Number of elements	40	80	-
Max. moment at the center of RBS at first floor (kN.m)	568.0	568.0	0.00
Max. moment at the face of column (kN.m)	646.7	647.0	0.05
Max. inter storey drift at first floor (%)	2.16	2.16	0.07
Max. inter storey drift at second floor (%)	1.95	1.95	0.20
Max. inter storey drift at third floor (%)	2.33	2.34	0.43
Max. inter storey drift at roof (%)	2.42	2.42	0.27
Max. roof acceleration (g)	0.44	0.44	0.00
Max. base shear (kN)	378.1	378.9	0.21

Figure 3.1: Amplification factor of isotropic yield stress calculated from Eq. (3.1) for materials H and G at a few strain rates

Figure 3.2: Amplification factor of flow stress $\sigma_t^F(\dot{\epsilon})/\sigma_t^F(0)$ calculated from Eq. (3.4) for materials H and G at a few strain rates

Figure 3.3: Isotropic stress versus true plastic strain curves at a few strain rates for materials H and G

Figure 3.4: Back stress versus true plastic strain for materials H and G

Figure 3.5: Flow stress (true stress) versus true plastic strain curves at a few strain rates for materials H and G

Figure 3.6: Building plan dimensions (Jin and El-Tawil 2004)

Figure 3.7: Elevation view of four-storey MRF by Jin and El-Tawil 2004

Figure 3.8: Plan of the building and locations of moment resisting frames

Figure 3.9: Elements, cross-section sizes and dimensions of the frames

Figure 3.10: Dimensions of RBS

Figure 3.11: RBS profile with varying width segments of beam flange

Figure 3.12: Mesh scheme at RBS

Figure 3.13: Mesh scheme for single storey MRF

Figure 3.14: Mesh scheme for four-storey MRF MD1

Chapter 4: Material and model validation with deep column beam moment connection with RBS

Post Northridge earthquake, new design recommendations including the addition of reduced beam section (RBS) has been put forward to avoid fracture in the beam column connection during a seismic event. In order to achieve an economical design for the moment resisting frames, deep columns are often used in the design to control the drift limit of the system. Most tests on moment resisting frames with RBS were conducted under static loading due to the complexity and challenge of conducting dynamic tests. Numerous experiments of RBS under static cyclic loading have been conducted by Chen et al. (1996), Plumier (1997), Zekioglu et al. (1997) and Engelhardt (1998).

Strain rate dependent material properties for cyclic loading calibrated by Singh (2019) presented in Chapter 3 are to be used in the numerical simulations of moment resisting frames with RBS. However, these material properties have not been validated against test results of RBS moment connections. There is a lack of results of RBS moment connections tested under dynamic cyclic loading. Thus, results of tests under quasi-static cyclic loading are considered instead for the validation. Among others, Chi and Uang (2002) conducted experiments to study the cyclic behavior in RBS moment connections with deep column under quasi-static loading. Analyses can be carried out to validate the suggested material properties against the test results of Chi and Uang (2002).

4.1 Test setup

Figure 4.1 represents the test setup experiment conducted by Chi and Uang (2002) for three full-scale specimens under cyclic loading. Specimen designation and member

sizes are represented in Table 4.1. The moment connection with RBS is designed according to Engelhardt (1998) and Seismic Provisions for Structural Steel Buildings (AISC 1997). Mechanical properties of ASTM A992 steel for the structural members obtained from the tension coupon tests are summarized in Table 4.2. The setup shown in Fig. 4.1 was tested under the standard SAC loading history (Clark et al. 1997) as shown in Fig. 4.2, applied at the end of the cantilever beam. Tests showed that specimens DC-1 and DC-2 experienced extensive lateral torsional buckling while DC-3 having a beam with larger out-of-plane to in-plane stiffness ratio and warping constant compared to that in DC-1 and DC-2, experienced much smaller lateral torsional buckling. Thus, only test results of DC-3 are used in the validation as there are large uncertainties in simulating inelastic lateral torsional buckling. The dimensions of RBS for DC-3 are shown in Fig. 4.3.

4.2 Finite element modelling

Two finite element models are considered for the test. One model consists of beam element only with B31OS beam element and hybrid model with a combination of S4R shell element at the RBS region and B31OS beam element for the rest of the frame. B31OS is a two-node linear 3D beam element with open section and bi-linear S4R is a four node shell element with reduced integration. In the beam element only model, the RBS region is divided in 40 elements with equal element size of 15.25 mm having varying flange width along the length calculated according to Eq. (3.7) and the rest of the beam has an average element size of 52 mm. The column is divided in 40 elements with the element size of 95.3 mm. The schematic diagram for the beam element only model is shown in Fig. 4.4. For the hybrid model, the RBS region of S4R shell element is divided into 40 elements in the

longitudinal direction. The web and flange of the cross-sectional area is equally divided into 28 elements and 10 elements respectively. The rest of the beam and column are modelled with B31OS beam element. The column consists of 40 elements with the element size of 95.3 mm and the rest of the beam except RBS is divided into 52 elements with the average element of size 52 mm. The dimensions and details of the RBS radius cut for the chosen model is shown in Fig. 4.3. The schematic for the hybrid model and its representation in ABAQUS is shown in Figs. 4.5 and 4.6 respectively. No mesh refinement or sensitivity study has been carried out for these two models.

In the hybrid model, the constraints between the nodes of shell elements and shell to beam element connection are imposed following Singh (2019). Constraints between nodes of shell elements are done with the help of multiple point constraints (MPC). Shown in Fig. 4.7 the nodes of the shell elements in the flange region of the shell element are constrained with MPC SLIDER, restraining its movement in straight line. Furthermore, in order to achieve the same amount of displacement in vertical and longitudinal directions, the corner nodes and web-flange intersection nodes are linearly constrained using *EQUATION. Similarly, the nodes in the web region are constrained using MPC BEAM, as shown in Fig. 4.7 (b). KINEMATIC COUPLING is applied at the end nodes of flange and central node of the web in order to have the same rotation on those nodes. The node at the center of the web is then connected to the beam element using MPC TIE as indicated in Fig. 4.7 (c).

No complete material properties of true stress-true strain curves suitable for calibrating the inputs for a constitutive model under cyclic loading has been provided by Chi and Uang (2002). Thus, mixed-mode hardening material model proposed by Lemaitre

and Chaboche (1990) and modified by Singh (2019) are used in the numerical simulations. Singh (2019) has calibrated the parameters for the model for materials H and G by Chen (2010) and Walker (2012). Materials H and G have yield strengths 373 MPa and 342MPa respectively. However, these yield strengths do not match that of the specimens in the tests by Chi and Uang (2002), as listed in Table 4.2. Therefore, the isotropic yield stress of the yield stress versus true strain curves for materials H and G are scaled by a constant factor such that the scaled initial yield stress matches that of the material in the test. For the shell element in the hybrid model, the yield stress for the flange and web are scaled independently and the scaled yield stress for the flange is applied to the whole section of the beam element. The scaled isotropic hardening yield stress versus true plastic strain curves for the flange and web of materials H and G are shown in Fig. 4.8. Parameters for kinematic hardening calibrated by Singh (2019) for materials H and G shown in Table 3.1 are used in the numerical simulations together with the scaled yield stress.

4.3 Results of analysis

Comparisons of the test and predicted beam bending moment versus rotation for DC-3 (Chi and Uang 2002) with materials H and G are shown in Fig. 4.9 for hybrid model and Fig. 4.10 for beam element only model. The beam bending moment is taken at the face of the column. There is a good agreement between the test and predicted moment versus rotation curves for the hybrid model where the maximum test moment at each loading cycle is within that for materials H and G up to a rotation of around 0.02. Beyond rotation cycle of 0.02, the predicted moment and rotation curves diverge from that of the test as the test started to experience lateral torsional buckling. This shows the mixed-mode hardening

material model used with properties and parameters of materials H and G can give a good prediction of cyclic behavior of RBS. It can also be seen in Table 4.3 where the predicted peak moments of 102% and 102% at rotations of 0.01 and 0.02 for material H, and 97% and 97% at around rotations of 0.01 and 0.02 for material G.

Overall, the beam element only model is stiffer than the hybrid model. Unlike the hybrid model, the beam element only model overpredicts the bending moment of the test with both material H and G, as can be seen in Fig. 4.10 especially at the elbow of the bending moment versus rotation curves, and in Table 4.3. However, the over prediction reduces as the loading cycle increases. This is due to the isotropic yield stresses of materials H and G increases sharply at the initial plastic strain as shown in Fig. 4.8. Thus, the difference in the isotropic yield stress for a small difference in the plastic strain is higher at the start of the plastic deformation. This difference diminishes with the increase in plastic deformation. Figures 4.11 and 4.12, and Table 4.4 compare the predicted bending moment of hybrid model and beam element only model. The difference between the predicted bending moment by the two models decreases as the loading cycle increases. This difference reduces to no greater than 4% in the predicted peak bending moment at the loading cycle range of 0.03 radian rotation, as indicated in Table 4.3. Thus for computational efficiency and due to the small differences in the results at a large plastic deformation, RBS may be modelled with beam elements instead of shell elements for the non-linear dynamic analyses as noted by Singh (2019).

In the hybrid model, the flange and web of RBS have been assigned different material properties in the numerical simulations. Numerical simulations of test DC-3 have also been carried out with the hybrid model considering both the flange and web of RBS

having the same material. Figures 4.13 and 4.14 show that there is only a small difference between the predicted bending moment when modelled the whole RBS with the flange material instead of different materials for web and flange. Thus, the difference in the predicted bending moment of the hybrid and beam element only models is not due to the flange and web of the RBS being modelled as separate materials in the hybrid model while only the flange material in the beam element only model.

Table 4.1: Specimen designation and member sizes (Chi and Uang 2002)

Specimen	Beam size	Column size
DC-1	W 920x223 (W 36x150)	W 690x217 (W 27x146)
DC-2	W 920x223 (W 36x150)	W 690x289 (W 27x194)
DC-3	W 690x289 (W 27x194)	W 690x289 (W 27x194)

Table 4.2 : Mechanical properties of the steel specimens (Chi and Uang 2002)

Specimen	Member	Coupon	Yield strength (MPa)	Tensile strength (MPa)
DC-1	Column W 690x217	Flange	358	474
		Web	344	433
	Beam W 920x223	Flange	400	455
		Web	359	445
DC-2	Column W 690x289	Flange	427	453
		Web	435	449
	Beam W 920x223	Flange	400	455
		Web	359	445
DC -3	Column W 690x289	Flange	427	453
		Web	435	449
	Beam W 690x289	Flange	427	453
		Web	435	449

Table 4.3: Comparisons of the test and predicted maximum bending moment

Around rotation cycle	Maximum moment					
	0.01 rad		0.02 rad		0.03 rad	
	Max. moment MN.m	$\frac{\text{Analysis}}{\text{Test}}$	Max. moment MN.m	$\frac{\text{Analysis}}{\text{Test}}$	Max. moment MN.m	$\frac{\text{Analysis}}{\text{Test}}$
Test	3.70	-	4.15	-	4.05	-
Hybrid model mat. G	3.60	0.97	4.05	0.97	4.31	1.06
Hybrid model mat. H	3.76	1.02	4.25	1.02	4.53	1.12
Beam only model mat. G	3.78	1.02	4.18	1.01	4.37	1.08
Beam only model mat. H	4.06	1.10	4.50	1.08	4.70	1.16

Table 4.4: Comparison of maximum bending moment of beam element only model with the hybrid model

Around rotation cycle	Maximum moment					
	0.01 rad		0.02 rad		0.03 rad	
	Max. moment MN.m	$\frac{\text{Beam}}{\text{Hybrid}}$	Max. moment MN.m	$\frac{\text{Beam}}{\text{Hybrid}}$	Max. moment MN.m	$\frac{\text{Beam}}{\text{Hybrid}}$
Hybrid model mat. G	3.60	1.05	4.04	1.03	4.31	1.01
Beam only model mat. G	3.78		4.18		4.37	
Hybrid model mat. H	3.76	1.08	4.25	1.06	4.53	1.04
Beam only model mat. H	4.06		4.50		4.70	

Figure 4.1: Test setup of RBS moment connection by Chi and Uang (2002)

Figure 4.2: Standard SAC loading history (Chi and Uang 2002)

Figure 4.3: Dimensions of RBS for DC-3

Figure 4.4: Schematic of beam element only model for DC-3

Figure 4.5: Schematic of the hybrid model for DC-3

Figure 4.6: Representation of the hybrid model in ABAQUS.

Figure 4.7: Shell to beam element connection (a) Nodes constraint on the flange of shell element (b) Nodes constraint on the web of shell element and (c) Shell to beam element connection (Singh, 2019)

Figure 4.8: Scaled isotropic yield stress versus true plastic strain for materials H and G

Figure 4.9: Comparisons of moment versus rotation of test DC-3 (Chi and Uang 2002) with hybrid model

Figure 4.10: Comparisons of moment versus rotation of test DC-3 (Chi and Uang 2002) with beam element only model

Figure 4.11: Comparisons of moment versus rotation of hybrid and beam element only model for material H

Figure 4.12: Comparisons of moment versus rotation of hybrid and beam element only model for material G

Figure 4.13: Comparisons of moment versus rotation of hybrid model with RBS web and flange having same and different material for material H

Figure 4.14: Comparisons of moment versus rotation of hybrid model with RBS web and flange having same and different material for material G

Chapter 5: Numerical simulations of MRF with RBS

A non-linear dynamic analysis of a building can be carried out by subjecting it to the ground motion that has a response spectrum matching that of the building site. Among other things, this analysis can predict the displacement, base shear, bending moments at various sections of the building, particularly at the center of RBS and at the beam-column moment connection. Based on the studies conducted by Atkinson and Adams (2013) and Halchuk et al. (2015), and incorporated in National Building Code of Canada (NBCC 2015), the analyses use the mean hazard level seismic ground motions that has a probability of 2% exceedance in 50 years. However, there is no recorded ground motion that is compatible to the prescribed hazard level at a site. In addition, some sites such as southern British Columbia may experience from more than one source or type of earthquake that contributes to the ground motion hazard.

Since there is a lack of suitable earthquake records available, existing recorded earthquake ground motions are modified to produce the response that matches the target response spectrum of the site. The modification is done by scaling the earthquake ground motion. Non-linear dynamic analyses of the designed 4-storey building in Chapter 3 subjected to the scaled ground motions have been carried out to study effects of strain rate dependency of material properties on the response of the structure for the inter-storey drift, base shear, bending moments and other important factors at various locations of the structure. Strain rate dependent material properties listed in Chapter 3 for materials H and G calibrated by Tarundeep (2019) have been used in the analyses.

5.1 Dynamic non-linear response history analysis

Buildings for moderately ductile MRF and low ductile MRF from Chapter 3 are subjected to a suite of scaled earthquake ground motions to carry out non-linear time history analyses. The structure is assumed to be constructed of steel with either material properties of material H or material G. Materials H and G have a yield strength of 373 MPa and 342 MPa respectively, which are close to the probable yield strength of 385 MPa for CSA 350W steel and its minimum yield strength of 350 MPa. Each frame with strain rate and non-strain rate dependent material properties is subjected to a suite of earthquake ground motions consisting of 11 seismic records. A damping ratio of 3% is considered in the calculation for Rayleigh damping. Variables α and β in Eq. (3.14) are calculated for the first two frequencies from the eigenvalue analysis of the frame.

The effect of ground motion on the first mode of the structure is considered to be the most impactful. However, the effect on a higher mode of the structure has also been considered for moderately ductile MRF (MD1) and limited ductility MRF (LD1). These frames are subjected to a suite of earthquake records scaled according to its second mode obtained from the eigenvalue analysis. For these cases, frequencies of the second and third mode are used in the calculation of Rayleigh damping. The mass participation factors for MD1, MD2 and LD1 are 9.86×10^{-1} , 9.93×10^{-1} and 9.81×10^{-1} for mode 1 and 1.38×10^{-2} , 0.65×10^{-2} and 1.51×10^{-2} for mode 2 respectively. A total of 176 analyses have been performed for various parametric combinations, which consist of different material properties, ductility, strain rate and non-strain rate dependent material properties, and vibration mode of the structure. Figure 5.1 shows the matrix of the parametric combinations

considered in the non-linear dynamic analyses.

5.2 Scaling of earthquake records

Existing ground motion records of a site that are compatible with its target response spectrum are known as site-specific earthquake records. However, due to the lack in availability of these site-specific earthquake records, modified records from other sites have been used in the analyses. These records can be modified with either linear scaling technique or spectral matching technique.

The linear scaling technique involves applying a scaling factor to the ground motion so that the response matches the target spectrum without changing the frequency content of the ground motion. Some of the most commonly employed of linear scaling techniques are peak ground acceleration (PGA) scaling and Applied Technology Council (ATC) scaling. In PGA scaling, the ground motion is multiplied with a factor such that PGA of scaled ground motion at a frequency matches that of the target spectrum. All the scaled ground motions have the same PGA of that frequency. However, the response spectrum of each ground motion is different (Amirzehni et al. 2015). The second method, ATC scaling is a two-step scaling technique adopted from FEMA P695 (FEMA 2009). In the first step, the variability between earthquake records in terms of magnitude, site conditions, distance of source and source type is normalized by matching the peak ground velocity (PGV) of individual ground motions at a frequency to the mean PGV of the suite of ground motions. In the second step, a scaling factor is calculated by matching the median of spectral acceleration of the earthquake suite at the fundamental time period of the structure to the spectral acceleration of the design spectrum according to

$$F_A = \frac{S(a)_T}{S(a)_{median}} \quad (5.1)$$

where $S(a)_T$ is the spectral acceleration of the design spectrum at the fundamental time period of the structure and $S(a)_{median}$ is the mean spectral acceleration of the earthquake suite. This scaling factor is applied to the individual acceleration time histories.

The other method is spectral matching technique which involves scaling the existing ground motion record with a constant scaling factor either in frequency domain or time domain, such that its response spectrum approximately matches the design spectrum of the site over a range of time period (Abrahamson 1992).

According to the records and complex tectonic plate arrangement, the ground motions on southwest British Columbia are associated to three categories of crustal, sub-crustal and subduction earthquakes. Crustal earthquakes are usually experienced in North America with the epicenter less than 35 km from the ground and magnitude as large as 7.5 (PNSN 1969). Subcrustal earthquakes are recorded to have epicenter greater than 35 km and magnitude less than 7.0. Sometimes at depth shallower than 30 km, the subduction earthquake can occur that has the potential to produce an earthquake of magnitude 9.0. Crustal and sub-crustal earthquakes have a greater impact on structures with a time period less than 0.7s, while structures having time period equal or greater than 1.5s are more impacted by subduction earthquakes.

There are many scaling procedures used to generate ground motions that can produce the response spectrum that matches the design spectrum of a site. National Building Code of Canada 2015 (NBCC 2015) has prescribed a procedure that is based on spectral matching to scale the earthquake records. In this procedure, a time period range (T_R) of the building is selected based on the periods of the vibration modes that contribute

to the dynamic response of the building, either in the translational direction and/or in torsion. As shown in Fig. 5.2, the upper bound ($T_{\max}$) of T_R should be greater than or equal to two times of the first mode time period. However, it should not be less than 1.5s. The lower bound ($T_{\min}$) of T_R should be the structural period at and above which at least accumulated 90% of mass participation has contributed, but not less than 0.15 times the first mode period. Mass participation factor is the fraction of total mass participation at a particular mode. Every mode is represented with its corresponding mass participation, and the mode with a large factor plays a vital role in defining the response of the building. The limits can be expressed as

$$T_{\max} = \max (2.0T_1, 1.5s) \quad (5.2)$$

$$T_{\min} = \min (0.15T_1, T_{90\%}) \quad (5.3)$$

In certain cases, there are more than one potential tectonic sources contributing to the total seismic hazard of that particular location. Thus, it becomes necessary to deaggregate these seismic hazard results to identify the dominant magnitude-distance (M-R) scenarios of the earthquake for the site. This requires the formation of more than one target spectrum between $T_{\min}$ and $T_{\max}$ that is generated for each M-R scenario. Each M-R scenario is assigned a segment of the period range (T_{RS}), which is called scenario-specific period range, T_{RSi} ($i= 1, 2, \dots$). According to the guidelines of NBCC 2015, the scaling of ground motion suites to match the target design spectrum can be achieved through two methods, Methods A and B.

In the first method (Method A), one target response spectrum, $S_T(T)$ corresponds to the design spectrum $S(T)$ is used over the whole period range T_R as shown in Fig. 5.3. For places where the hazard is contributed by more than one tectonic source, at least one

scenario-specific period range must be defined for each tectonic source. The period range T_R can be divided in scenario-specific period range T_{RS} such that it covers the entire period range T_R . The sub-period ranges T_{RS} can overlap each other, but should cover the entire period range.

For Method B, two or more scenario-specific target response spectra $S_T(T)$ must be defined over the period range T_R as shown in Fig. 5.4. Selection of ground motion should be based on earthquake magnitude and distance combinations as well as tectonic sources that are consistent with the scenario target spectra. Method B can be further divided into Method B1 and Method B2. For Method B1, a site-specific target spectra $S_T(T)$ are created for all the dominating earthquake magnitudes and tectonic sources that contribute to the hazard over the target period range T_R . These dominant sources and their corresponding sub-period range T_{RS} can be known from seismic hazard disaggregation of the site. At least one target spectrum from each tectonic source contributing to the hazard is required to create the target spectra envelope. This envelope should not be less than the design spectrum $S(T)$ over the period range T_R .

For Method B2, the site-specific target spectrum is created for the time period for which the vibration modes contribute significantly to the dynamic response of the building. Similar to Method B1, the target spectra for each hazard dominant earthquake magnitude-distance combination at each time period can be obtained from site-specific disaggregation. The guidelines suggest using the ground motion prediction equations (GMPEs) to define the spectral shapes. The scenario-specific target spectra envelope should not be less than 75% of the design spectrum over the period range T_R as shown in Fig. 5.4 (a).

For Method A, there should be a minimum of 5 ground motion records in each suite

for the respective scenario-specific time period ranges (T_{RS}) of the target design spectrum. However, the total number of earthquake records in these suites should not be less than 11. For Method B, at least 11 earthquake records should be chosen to match each scenario-specific target spectrum.

In these methods, ground motions are scaled in two stages to match the target spectrum. The first stage involves scaling each ground motion in the suite individually such that it roughly matches the target response spectrum $S_T(T)$ over the specified period range T_R . A linear regression analysis is employed to match the spectral acceleration of individual earthquake record with the target spectrum. Method of least square error is used to optimize the scaling by minimizing E_r in

$$r_i = \alpha_f y_i - x_i \quad (5.4)$$

$$E_r = \sum_i^n r_i^2 \quad (5.5)$$

where y_i and x_i are spectral accelerations for the response spectrum and target spectrum respectively at a given time period i , α_f is the scaling factor, n is the total time interval of the scenario-specific earthquake record, r_i is the difference and E_r is the sum of squares of the difference. Optimum scaling factor is obtained when E_r is minimum.

The second stage is to scale each suite by a second common factor. In this stage, the mean of spectral acceleration of the entire suite is scaled through linear regression analysis to match it with the target spectrum. To optimize the results, least square method is used to optimize the scaling by minimizing E_r in

$$rs_{ij} = \beta_g \alpha_{f_j} y_i - x_i \quad (5.6)$$

$$E_r = \sum_{j=1}^m \sum_{i=1}^n (rs_{ij})^2 \quad (5.7)$$

where β_g is the scaling factor for the entire suite, α_{f_j} is the scaling factor of the individual ground motion, m is the total number of ground motions in the suite, rs_{ij} is the difference and E_r is the sum of the squares of the difference. However, the mean response spectrum of the suite should not be 10% below the target spectrum $S_T(T)$. The factor β_g from optimization is used in calculating the mean spectrum of the suite. Also, it should be taken into consideration that if the final ground motion scaling factor ($\beta_g \alpha_{f_j}$) is less 0.5 or larger than 4.0, it might not be compatible with the considered earthquake hazard level. In such situation, the selections of ground motion must be revised.

For multidirectional analysis, the pair of horizontal components ground motion records should be scaled with a single scaling factor calculated according to stage 1 from the Eqs. (5.4) and (5.5). The final scaling factor $\beta_g \alpha_{f_j}$ should be taken such that the geometric mean of the response spectrum of the two horizontal components matches the design response spectrum.

For simplicity, only scaling Method A in NBCC 2015 is used in this study since the main objective of this research is to assess the effects of material properties strain rate dependency. In addition, the more sophisticated Method B1 and B2 requires the use of an elaborated seismic hazard disaggregation method such as GMPE. Thus, Method B1 and B2 are not considered in this study.

A suite of earthquake records shown in Table 5.1 are used in this study. Calculated from Eqs. (5.2) and (5.3), and shown in Fig. 5.5, the target spectrum limit is 0.22s to 2.9s for the moderately ductile MRFs (MD1). Not shown, the calculated target spectrum limits for moderately ductile MRF (MD2) and limited ductility MRF (LD1) that are 0.26s to 3.5s and 0.18s to 2.48s respectively. The individual scaling factor (α_{f_j}) for each earthquake

record for moderately ductile MRFs (MD1 and MD2) and limited ductility MRF (LD1) ranges from 0.58 to 2.91, 0.67 to 2.88 and 0.70 to 2.92. Figure 5.6 shows the earthquake record suite scaled individually for moderately ductile MRF (MD1). The suite scaling factors (β_g) are 1.28, 1.35 and 1.22 for MD1, MD2 and LD1 respectively. Figure 5.7 shows the scaled earthquake record suite for moderately ductile MRF (MD1). The final scaling factor ($\beta_g \alpha_{f_j}$) for each earthquake record for moderately ductile MRFs (MD1 and MD2) and limited ductility MRF (LD1) ranges from 0.73 to 3.72, 0.91 to 3.89 and 0.85 to 3.56 respectively.

5.3 Numerical simulations

A suite of 11 earthquake records shown in Table 5.1 have been scaled and used to perform the numerical simulations of four storey MRFs with RBS. Moderately ductile and limited ductility MRFs are assessed for effects of strain rate dependency of material properties for bending moment, base shear, inter storey drift and strain hardening. Both materials H and G are considered in the numerical simulations.

5.3.1 Strain rate

Chang and Lee (1987) stated that a building may experience a strain rate at up to 10^{-1}s^{-1} during an earthquake. At this strain rate level, the strain rate dependency of the material properties can have significant effects on the performance of the building and on the forces induced in the structure due to the ground excitation. In the MRF with RBS, the maximum strain rate is expected to occur at the center of RBS since the design specifically localized the plastic deformation to this region. Maximum strains occur at the top and

bottom flanges at the center of RBS. Thus, the maximum strain rate is calculated from the larger rate of these two flanges. Tables 5.2 to 5.7 show the maximum strain rate at the center of RBS at each floor level for both moderately ductile (MD1 and MD2) and limited ductility (LD1) MRFs.

For the moderately ductile MRF (MD1) as shown in Table 5.2, the first floor experiences a maximum strain rate range of 0.07s^{-1} to 0.11s^{-1} and 0.07s^{-1} to 0.13s^{-1} for materials H and material G respectively. The ranges for the second floor increase to 0.08s^{-1} to 0.13s^{-1} for material H and 0.09s^{-1} to 0.14s^{-1} for material G. As shown in Table 5.3, the maximum strain rate is 0.16s^{-1} at the third floor for both materials H and G, and a maximum of 0.18s^{-1} at the roof. The maximum strain rate appears to larger at a higher floor. However, it does not appear to be strongly dependent on the material properties as the maximum strain rates are almost the same for both materials H and G.

Shown in Table 5.4 is the maximum strain rate experienced at the center of RBS for moderately ductile MRF (MD2) for each earthquake ground motion on first and second floor. For the first floor, the maximum strain rate ranges from 0.09s^{-1} to 0.12s^{-1} and 0.09s^{-1} to 0.14s^{-1} for materials H and G respectively. The maximum strain rate increases slightly at the second floor for ranges of 0.10s^{-1} to 0.14s^{-1} and 0.10s^{-1} to 0.15s^{-1} for materials H and G respectively. As shown in Table 5.5, the maximum strain rate for both materials H and G at the third floor and roof are 0.16s^{-1} and 0.18s^{-1} respectively. Similar to MD1, a same trend of increase in strain rate at a higher floor can be observed for moderately ductile MRF (MD2). Also, the rate appears not to be dependent on the material properties as there is not substantial difference between values for materials H and G.

Results of the maximum strain rate at the center of RBS for limited ductility MRF

(LD1) are shown in Tables 5.6 and 5.7. The maximum strain rate ranges are 0.07s^{-1} to 0.10s^{-1} and 0.07s^{-1} to 0.12s^{-1} for materials H and G respectively, at the first floor and 0.10s^{-1} to 0.15s^{-1} , and 0.10s^{-1} to 0.14s^{-1} at the second floor. These ranges are comparable to that for moderately ductile MRFs (MD1 and MD2). Maximum strain rate at the third floor and the roof are 0.17s^{-1} and 0.19s^{-1} , which are also comparable to that for the moderately ductile MRF (MD1 and MD2). There is also a similar increase in the maximum strain rate with height and the lack of dependency on material properties as observed for moderately ductile MRFs (MD1 and MD2). For all three frames, the mean maximum strain rate at each floor is close to or greater than 0.1s^{-1} . Thus, effects of strain rate dependency of the material properties can be important. However, the maximum strain rate experienced does not appear to be influenced by the material properties nor the designed ductility level of the MRF with RBS.

5.3.2 Bending moment

The strain rate dependency of the material properties affects moments generated in the beam, particularly at the center of RBS, where the largest strain and strain rate are expected to occur during an earthquake event. Bending moments at the center of RBS, face of column and centerline of column are assessed and compared for effects of strain rate dependency of material properties. The probable moment is calculated according to the provisions mentioned in CSA S16-14 (CSA 2014) and guidelines from Moment Connections for Seismic Application (CISC 2014). The probable moments at various locations for frame MD1, MD2 and LD1 are shown in Table 5.8. Tables 5.9 to 5.17 show bending moments of frames MD1, MD2 and LD1 for materials H and G with strain rate

dependent and independent material properties respectively.

Table 5.9 shows the mean predicted bending moment generated at the center of RBS for moderately ductile MRF (MD1) with materials H and G. The moment increase ranges from 5.6% to 6.9% due to the effect of strain rate for material H, and 8.4% to 8.8% for material G. As expected, material G has a higher isotropic yield stress amplification with strain rate compared to material H. Overall, the predicted static moment with material G is only slightly higher than the probable moment (1.01 to 1.02), while the predicted static moment with material H is about 4% higher (1.05 to 1.06) than with material G since material H has a higher yield strength than material G. Even though material H has a 9% higher yield strength than material G (373 MPa versus 343 MPa), the predicted moment with material H is only around 4% higher because material G has a higher initial rate of strain hardening strength increase after yielding. With the strain rate dependent material properties, the predicted moment with material G is considerably higher (10%) than the probable moment (1.10 to 1.11), while the predicted moment with material H is about 2% higher than that with material G (1.12 to 1.13). The difference between the predicted moment for materials H and G from without to with strain rate dependency in the material properties reduces since material G has a higher isotropic yield stress amplification. Trends similar to the predicted moment at the center of RBS can be observed in the predicted moment at the face and centerline of the column for moderately ductile MRF (MD1) in Tables 5.10 and 5.11. Figures 5.8 and 5.9 show the predicted maximum moment at the center of RBS for the first floor and roof at the center of RBS for moderately ductile MRF (MD1). The predicted moments are higher than the probable moments under all ground motions except for a few ground motions with static material properties of material G.

Table 5.12 shows the predicted bending moment at the center of RBS for moderately ductile MRF (MD2) with materials H and G. The moment increases by 7.4% to 8.5% due to effects of strain rate for material H, and 8.8% to 9.8% for material G. These increases are slightly higher than those obtained for moderately ductile MRF (MD1). Here, the predicted static moment is slightly higher than the probable moment (1.00 to 1.02) for material G and the predicted static moment for material H is about 3% higher (1.02 to 1.05) than material G. Similar to results for moderately ductile MRF (MD1), the predicted moment at the center of RBS for material G with strain rate dependent material properties increased by 9% (1.09 to 1.12) when compared to the corresponding probable bending moment. The predicted moment with material H is only slightly higher than with material G (1.10 to 1.13). Figures 5.10 and 5.11 show the predicted bending moment at the center of RBS for the first floor and roof for moderately ductile MRF (MD2). Except for a small number of ground motion with static material properties for materials H and G, the predicted bending moments are higher than the probable moments under other ground motions and with strain rate dependent material properties. Tables 5.13 and 5.14 show the mean predicted bending moments at the face and centerline of the column for moderately ductile MRF (MD2). These results show similar trends as the moment at the center of RBS.

Table 5.15 shows the mean predicted bending moment generated at the center of RBS for limited ductility MRF (LD1) with materials H and G. The moment increases at 4.7% to 6.3% due to effects of strain rate for material H, and 6.8% to 7.8% for material G. These increases are slightly lower compared to that for moderately ductile MRF (MD1). The predicted static moment with material G is about same as the probable moment (0.99 to 1.04), while the predicted static moment with material H is around 3% higher (1.03 to

1.07) than that with material G. Similar to moderately ductile MRF (MD1), the difference between the predicted moment with materials H and G is much less than the actual difference in the yield strength. With the strain rate dependent material properties, the predicted moment with material G is about 8% (1.07 to 1.12) higher than that of the static moment, while the predicted moment with material H is about 1% higher compared to that with material G (1.08 to 1.13). The predicted moments at the face and centerline of the column follow the same trends as the moment at the center of RBS. Figures 5.12 and 5.13 show the predicted maximum moment on the first floor and the roof at the center of RBS for limited ductility MRF (LD1). The predicted bending moments are higher than the probable moments under all ground motions except for a small number of ground motions with material G.

The increase in the predicted moment at the center of RBS with the consideration of strain rate dependency of material properties over that of the static moment ranges for 5.6% to 8.8% for moderately ductile MRF (MD1), 7.4% to 9.4% for moderately ductile MRF (MD2), and 4.7% to 7.8% for the limited ductility MRF (LD1). The rate of increase is higher for MRF MD2 as the frame has been designed to be more flexible. The underestimation of the bending moment at the center of RBS is up to 7% with static material properties and 13% with strain rate dependent material properties. Results of predicted maximum moment at the center of RBS for individual ground motion are shown in Appendix C. The rate of increase due to material strain rate dependency can be as high as 12% with material H and 15% with material G for some scaled ground motions.

5.3.3 Maximum inter-storey drift

According to the provisions in National Building Code of Canada (NBCC 2015), the inter-storey drift for normal importance building should not exceed 2.5%. The moderately ductile MRF (MD2) has been designed only for strength has the inter-storey drift exceeded 2.5%. However, for moderately ductile MRF (MD1), the column has been oversized to bring the expected inter-storey drift down to around 2.5%.

Tables 5.18 to 5.29, and Figs. 5.14, 5.15 and 5.16, show the predicted inter-storey drift for moderately ductile MRF (MD1 and MD2) and limited ductility MRF (LD1) with strain rate dependency and independent (static) material properties for materials H and G. It can be seen that the strain rate dependency in material properties and difference in material strength do not have any appreciable effect on the predicted inter-storey drift. The largest difference between the mean predicted inter-storey drift at each floor between materials H and G is 0.02%, and the largest difference between with and without strain rate dependency in material properties is 1.2%. Overall, the mean predicted inter-storey drift with strain rate dependent material properties vary between 6.2% and 27.4% below the design inter-storey drift, as shown in Tables 5.30 to 5.32.

5.3.4 Maximum base shear

Base shear is the maximum lateral force acting at the base of a structure. The design base shear obtained from the equivalent static analysis method for the final design time periods of 1.45s, 1.75s and 1.24s are 321 kN, 275 kN and 467 kN for moderately ductile MRFs (MD1 and MD2) and limited ductility MRF (LD1) respectively.

The actual base shear generated depends on the bending moment developed at the

center of RBS. Table 5.33 shows bending moment from the equivalent static analysis and the probable moment at the center of RBS. The probable base shear can be estimated to be equal to the base shear from the equivalent static analysis scaled by the mean probable moment over the predicted moment from the equivalent static analysis over the four levels. The mean of the ratios are 1.36 for moderately ductile MRF (MD1), 1.35 for moderately ductile MRF (MD2) and 1.23 for limited ductility MRF (LD1). Multiplying these ratios with the corresponding base shear values from equivalent static analysis the absolute probable base shear obtained are 437 kN, 371 kN and 574 kN for MRFs MD1, MD2 and LD1 respectively. These base shear values are estimated based on the absolute maximum probable moment at the center of RBS occurring at the same time on all floors. However, this circumstance is unlikely to occur under any earthquake ground motion. Thus, the base shear obtained from equivalent static analyses of the frames is used for further comparisons.

Table 5.34 shows the mean predicted maximum base shear with static and strain rate dependent material properties for both materials H and G compared to the design base shear. There is an increase in the mean predicted maximum base shear with strain rate dependent material properties over that without strain rate dependency. For moderately ductile MRF (MD1), the increase is 3.0% with material H and 2.0% with material G. The increase in the mean predicted maximum base shear for moderately ductile MRF (MD2) is 3.2% with material H and 2.3% with material G, and 4.7% with material H and 3.5% with material G for limited ductility MRF (LD1). The limited ductility MRF (LD1) has higher (~ 1.5%) increase in the mean predicted maximum base shear due to the consideration of the strain rate dependency of material properties compared to moderately ductile MRFs.

Unlike the bending moment at the center of RBS, the increase in the mean predicted maximum base shear with material properties strain rate dependency is higher with material H than material G even though material G has a higher factor of isotropic yield stress amplification with strain rate. The actual increase in the mean predicted maximum base shear due to material properties strain rate dependency is a result of the complex interaction of responses of all the beams (RBSs) in addition to the strain rate amplification. Nevertheless, these increases are much smaller than the bending moment at the center of RBS. One possible reason material H has a higher rate of increase is higher strength. However, no further investigation has been carried out to explore why material H has a slightly higher increase.

As expected, the mean predicted maximum base shear with material H is higher than with material G since material H has higher yield strength. Compared to the probable base shear, the ratio of predicted to probable base shear for the moderately ductile MRF (MD1) is 1.14 for material H and 1.09 for material G with static material properties, and correspondingly 1.17 and 1.11 with strain rate dependent material properties. These values are 1.13, 1.08, 1.17 and 1.11 for moderately ductile MRF (MD2) and 1.14, 1.12, 1.20 and 1.15 for the limited ductility MRF (LD1). These are smaller than the mean ratios of the design over probable bending moment at the center of RBS of 1.36, 1.35 and 1.23 for MRFs MD1, MD2 and LD1. Thus, the connection at the support may be designed for force increases by a factor half the mean ratio of the design over probable moment at the center of RBS for a moderately ductile MRF and the mean ratio for a limited ductility MRF.

5.3.5 Strain hardening

Strain hardening occurs when the metal is strained beyond its yield point through plastic deformation. In the design of moderately ductile MRF (MD1), the strain hardening factor at the center of RBS is taken 1.1 due to significant plastic deformation at the plastic hinge. However, for limited ductility MRF (LD1), the strain hardening factor is taken 1.0 as there is less plastic deformation, and thus less strain hardening. The strain hardening factor can be calculated as

$$F_{sh} = \frac{M}{Z_x F_y} \quad (5.8)$$

where M is the moment, Z_x is the plastic section modulus and F_y is the initial static yield stress of the material. The static initial yield stress for materials H and G are 373 MPa and 342 MPa respectively.

Table 5.35 shows the predicted mean strain hardening factor calculated based on the predicted bending moment at the center of RBS with strain rate independent (static) material properties. The strain hardening factor with material G is higher than with material H since the rate of strength increase with strain hardening for material G is higher than material H. Moderately ductile MRFS (MD1 and MD2) also have about a 10% higher strain hardening factor compared to that for the limited ductility MRF (LD1), similar to the difference in the design factors of 1.1 and 1.0. The static mean predicted strain hardening factor for moderately ductile MRF (MD1) ranges from 1.19 to 1.20 with material H, and 1.25 to 1.26 for material G. For moderately ductile MRF (MD2) the static mean predicted strain hardening factor ranges from 1.15 to 1.19 for material H and 1.23 to 1.26 for material G. For the limited ductility MRF (LD1), the static strain hardening factor ranges from 1.06 to 1.10 for material H and 1.11 to 1.17 for material G.

With strain rate dependent material properties, the mean strain hardening factor for moderately ductile MRF (MD1) ranges from 1.26 to 1.28 with material H and 1.36 to 1.37 with material G. For moderately ductile MRF (MD2), the strain hardening factor ranges from 1.25 to 1.28 with material H and 1.34 to 1.38 with material G. The factor for limited ductility MRF (LD1) ranges from 1.11 to 1.16 with material H and 1.19 to 1.26 with material G. These results are shown in Table 5.36. The high ratio with strain rate dependent material properties is misleading because the ratio is calculated using static yield stress. The underestimation of the strain hardening factor ranges 5% to 16% with static and 15% to 28% with strain rate dependent material properties for moderately ductile MRFs (MD1 and MD2). For limited ductility MRF (LD1), the corresponding ranges are 6% to 17% and 11% to 26%. However, the underestimation of the bending moment at the center of RBS of the probable moment is more critical for the design compared to the strain hardening factor.

5.4 Other MRFs analyses

Besides analyses of MRFs carried out in Section 5.3, additional analyses have been carried out using earthquake records scaled for mode 2 time period and MRF with fixed supports. Appendix D provides details of a brief study on effects of earthquake records scaled for mode 2 time period instead of mode 1, which is the dominant mode of the frame. Analyses have been carried out for moderately ductile MRF (MD1) and limited ductility MRF (LD1) using earthquake records scaled for mode 2. Results of bending moment at the center of RBS and inter-storey drift are much smaller on an average of close to at least 10% with earthquake records scaled for mode 2 time period instead of mode 1. Thus, analyses

carried out with earthquake records scaled for mode 1 time period in Section 5.3 that produces a more severe response are adequate for studying effects of strain rate dependency of material properties.

Analyses in Section 5.3 have been carried using MRFs that are pinned supported. Appendix E provides details of a brief study of a moderately ductile MRF (MD3) with fixed supports. Results of the analyses show that the maximum rate of increase in the predicted maximum moment over the probable moment at the center of RBS and the difference in the predicted inter-storey drift compared to the design value are comparable between pinned and fixed supports MRFs MD1 and MD3. Thus, findings on the bending moment at the center of RBS and inter-storey drift for the pinned supported MRFs in Section 5.3 may be extended to fixed supported MRFs.

5.5 Concluding remarks

The analyses show that the frames can experience a strain rate higher than 1.0s^{-1} , which is at a rate where effects of strain rate dependency of material properties on the structural response can be significant. However, the maximum strain rate experienced does not appear to be influenced by the material properties nor the designed ductility level of the MRFs considered in this study. The material strength and strain rate dependency of material properties also do not appear to have appreciable effect on the maximum inter-storey drift (deflection) of the frames. Nevertheless, the mean predicted maximum inter-storey drift varies between 6% to 27% below the design inter-storey drift for the frames and material properties considered.

The strain rate dependency of the material properties has a significant effect on the

forces generated in the frames. At the center of RBS, the mean strain rate hardening factor with (static) non-strain rate dependent material properties is higher by around 10% for material H and 15% for material G over the prescribed factors of 1.1 and 1.0 for moderately ductile and limited ductility MRFs respectively. The factors with respect to the initial static yield stress increase around 7% for material H and 10% for material G with strain rate dependent material properties. In terms of bending moment at the center of RBS, the average increase in the bending moment varies from 5.5% to 8% for material H and 7.3% to 9.3% for material G from MRFs LD1, MD1 to MD2 when the strain rate dependency of material properties is considered. The maximum underestimation of the mean maximum bending moment at the center of RBS is up to 13%. While this underestimation may not cause the failure of beam to column connection (bolt or weld), it will reduce the safety factor of the design. Thus, the design forces for the connection are recommended to increase by a factor of 1.10.

Overall, the base shear is underestimated by around 14% based on static material properties and 20% with strain rate dependent material properties compared to the design base shear. The rate of increase due to material properties strain rate dependency is smaller for the base shear compared to the bending moment at the center of RBS. In order to maintain the safety factor for the connection at the support, the design force should be increased by a factor that is half the mean ratio of the design over probable moment at the center of RBS for a moderately ductile MRF and mean ratio for a limited ductility MRF.

Table 5.1: Selected ground motion records from PEER NGA-West 2 database (PEER 2013)

Earthquake Record (ER)	EQ ID (RSN)	Earthquake Event	Recording Station	Magnitude of Earthquake	PGA (g)
1	15	Kern County, 1952	Taft Lincoln School	7.36	1.03
2	78	San Fernando, 1971	Palmdale Fire Station	6.61	1.14
3	79	San Fernando, 1971	Pasadena CIT Athenaeum	6.61	1.17
4	136	Santa Barbara, 1978	Santabarbara courthouse	5.92	1.29
5	139	Tabas, Iran, 1978	Dayhook	7.35	1.49
6	150	Cayote Lake, 1979	Gilroy Array	5.74	0.95
7	164	Imperial Valley, 1979	Cerro Prieto	6.53	1.00
8	212	Livermore, 1980	Del Valley Dam	5.8	1.12
9	214	Livermore, 1980	San Ramon- Eastman Kodak	5.8	0.83
10	286	Irpinia, Italy, 1980	Bisaccia	6.9	1.36
11	289	Irpinia, Italy, 1980	Calitri	6.9	1.10

Table 5.2: Maximum strain rate (s^{-1}) at center of RBS on first and second floor with strain rate dependent material properties for moderately ductile MRF (MD1)

ER	First floor		Second floor	
	Material H	Material G	Material H	Material G
1	0.10	0.10	0.11	0.11
2	0.11	0.11	0.13	0.13
3	0.10	0.09	0.10	0.09
4	0.09	0.09	0.09	0.10
5	0.10	0.10	0.11	0.11
6	0.07	0.07	0.09	0.09
7	0.11	0.12	0.13	0.14
8	0.11	0.11	0.13	0.14
9	0.09	0.12	0.10	0.13
10	0.10	0.13	0.12	0.14
11	0.08	0.09	0.08	0.10
Mean	0.10	0.10	0.11	0.12

Table 5.3: Maximum strain rate (s^{-1}) at center of RBS on third floor and roof with strain rate dependent material properties for moderately ductile MRF (MD1)

ER	Third floor		Roof	
	Material H	Material G	Material H	Material G
1	0.12	0.13	0.15	0.16
2	0.15	0.15	0.16	0.16
3	0.14	0.13	0.14	0.14
4	0.10	0.11	0.11	0.11
5	0.12	0.12	0.14	0.15
6	0.10	0.10	0.12	0.14
7	0.16	0.16	0.17	0.18
8	0.15	0.15	0.18	0.18
9	0.12	0.14	0.13	0.15
10	0.10	0.15	0.14	0.16
11	0.13	0.14	0.16	0.17
Mean	0.13	0.13	0.14	0.15

Table 5.4: Maximum strain rate (s^{-1}) at center of RBS on first and second floor with strain rate dependent material properties for moderately ductile MRF (MD2)

ER	First floor		Second floor	
	Material H	Material G	Material H	Material G
1	0.11	0.12	0.11	0.11
2	0.10	0.11	0.14	0.14
3	0.10	0.10	0.12	0.10
4	0.09	0.10	0.10	0.10
5	0.11	0.11	0.12	0.13
6	0.09	0.09	0.10	0.11
7	0.12	0.14	0.14	0.15
8	0.11	0.11	0.13	0.14
9	0.11	0.13	0.11	0.14
10	0.11	0.13	0.12	0.15
11	0.09	0.10	0.10	0.11
Mean	0.10	0.11	0.12	0.13

Table 5.5: Maximum strain rate (s^{-1}) at center of RBS on third floor and roof with strain rate dependent material properties for moderately ductile MRF (MD2)

ER	Third floor		Roof	
	Material H	Material G	Material H	Material G
1	0.12	0.12	0.14	0.16
2	0.16	0.16	0.17	0.18
3	0.14	0.14	0.15	0.15
4	0.11	0.12	0.12	0.13
5	0.13	0.13	0.14	0.16
6	0.11	0.11	0.12	0.13
7	0.15	0.16	0.18	0.18
8	0.16	0.16	0.18	0.18
9	0.13	0.14	0.14	0.16
10	0.11	0.15	0.13	0.17
11	0.13	0.16	0.15	0.18
Mean	0.13	0.14	0.15	0.16

Table 5.6: Maximum strain rate (s^{-1}) at center of RBS on first and second floor with strain rate dependent material properties for limited ductility MRF (LD1)

ER	First floor		Second floor	
	Material H	Material G	Material H	Material G
1	0.07	0.07	0.09	0.10
2	0.10	0.10	0.15	0.14
3	0.08	0.09	0.10	0.11
4	0.08	0.10	0.09	0.10
5	0.10	0.10	0.11	0.11
6	0.07	0.07	0.09	0.08
7	0.09	0.11	0.11	0.12
8	0.10	0.12	0.11	0.12
9	0.10	0.14	0.10	0.15
10	0.09	0.11	0.13	0.14
11	0.09	0.10	0.08	0.11
Mean	0.09	0.10	0.10	0.12

Table 5.7: Maximum strain rate (s^{-1}) at center of RBS on third floor and roof with strain rate dependent material properties for limited ductility MRF (LD1)

ER	Third floor		Roof	
	Material H	Material G	Material H	Material G
1	0.11	0.12	0.13	0.13
2	0.17	0.15	0.19	0.16
3	0.15	0.16	0.18	0.18
4	0.13	0.13	0.16	0.16
5	0.13	0.13	0.16	0.16
6	0.13	0.12	0.15	0.13
7	0.16	0.16	0.19	0.19
8	0.15	0.17	0.16	0.18
9	0.14	0.15	0.17	0.18
10	0.15	0.15	0.16	0.16
11	0.12	0.12	0.13	0.13
Mean	0.14	0.14	0.16	0.16

Table 5.8: Probable moments calculated according to CSA S16-14 (CSA 2014) and Moment Connections for Seismic Application (CISC 2014)

Floor level	Moment at center of RBS, M_{pr} (kN.m)	Moment at face of column, M_{cf} (kN.m)	Moment at centerline of column, M_c (kN.m)
Moderately ductile MRF (MD1)			
First floor	520	569	628
Second floor	520	569	628
Third floor	184	195	221
Roof	96	104	120
Moderately ductile MRF (MD2)			
First floor	460	501	540
Second floor	449	489	528
Third floor	181	196	214
Roof	85	92	103
Limited ductility MRF (LD1)			
First floor	590	635	701
Second floor	590	635	701
Third floor	207	223	251
Roof	104	112	130

Table 5.9: Mean predicted maximum moment at the center of RBS with static and strain rate dependent material properties for moderately ductile MRF (MD1)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	544	582	6.90	1.05	1.12
	Mat. G	527	572	8.40	1.01	1.10
Second floor	Mat. H	549	586	6.70	1.06	1.13
	Mat. G	530	576	8.60	1.02	1.11
Third floor	Mat. H	195	207	6.00	1.06	1.12
	Mat. G	187	204	8.80	1.02	1.11
Roof	Mat. H	102	107	5.60	1.06	1.12
	Mat. G	97	106	8.80	1.01	1.10

Table 5.10: Mean predicted maximum moment at the face of column with static and strain rate dependent material properties for moderately ductile MRF (MD1)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{cf}}$	$\frac{M_r}{M_{cf}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	590	632	7.1	1.04	1.11
	Mat. G	571	620	8.6	1.00	1.09
Second floor	Mat. H	595	636	6.9	1.05	1.12
	Mat. G	578	629	8.8	1.02	1.11
Third floor	Mat. H	206	220	6.8	1.06	1.13
	Mat. G	197	214.5	8.9	1.01	1.10
Roof	Mat. H	109	116	6.4	1.05	1.12
	Mat. G	105	114.5	9.0	1.01	1.10

Table 5.11: Mean predicted maximum moment at the centerline of column with static and strain rate dependent material properties for moderately ductile MRF (MD1)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_c}$	$\frac{M_r}{M_c}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	662	710	7.3	1.05	1.13
	Mat. G	640	697	8.9	1.02	1.11
Second floor	Mat. H	672	719	7.0	1.07	1.14
	Mat. G	645	703	9.0	1.03	1.12
Third floor	Mat. H	235	251	6.8	1.06	1.14
	Mat. G	225	245	8.9	1.02	1.11
Roof	Mat. H	124.5	133	6.8	1.04	1.11
	Mat. G	121	132	9.1	1.01	1.10

Table 5.12: Mean predicted maximum moment at the center of RBS with static and strain rate dependent material properties for moderately ductile MRF (MD2)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	470	510	8.5	1.02	1.11
	Mat. G	465	506	9.0	1.01	1.10
Second floor	Mat. H	457	494	8.1	1.02	1.10
	Mat. G	450	491	9.1	1.00	1.09
Third floor	Mat. H	190	204	7.4	1.05	1.13
	Mat. G	184	202	9.8	1.02	1.12
Roof	Mat. H	87	94	8.0	1.02	1.11
	Mat. G	85	93	9.4	1.00	1.09

Table 5.13: Mean predicted maximum moment at the face of column with static and strain rate dependent material properties for moderately ductile MRF (MD2)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{cf}}$	$\frac{M_r}{M_{cf}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	515	560	8.7	1.03	1.12
	Mat. G	503	548	8.9	1.00	1.09
Second floor	Mat. H	502	544	8.4	1.03	1.11
	Mat. G	489	533	9.0	1.00	1.09
Third floor	Mat. H	204	221	8.3	1.04	1.13
	Mat. G	198	217	9.6	1.01	1.11
Roof	Mat. H	94.4	102	8.1	1.03	1.11
	Mat. G	91	99.5	9.3	0.99	1.08

Table 5.14: Mean predicted maximum moment at the centerline of column with static and strain rate dependent material properties for moderately ductile MRF (MD2)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_c}$	$\frac{M_r}{M_c}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	550	598	8.7	1.02	1.11
	Mat. G	538	586	8.9	1.00	1.09
Second floor	Mat. H	542	587	8.3	1.03	1.11
	Mat. G	528	575	8.9	1.00	1.09
Third floor	Mat. H	222	240	8.1	1.04	1.12
	Mat. G	216	236.5	9.5	1.01	1.11
Roof	Mat. H	105	113.5	8.1	1.02	1.10
	Mat. G	101	110.5	9.4	0.98	1.07

Table 5.15: Mean predicted maximum moment at the center of RBS with static and strain rate dependent material properties for limited ductility MRF (LD1)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	622	661	6.3	1.05	1.12
	Mat. G	612	654	6.9	1.04	1.11
Second floor	Mat. H	627	664	5.9	1.06	1.13
	Mat. G	615	659	7.2	1.04	1.12
Third floor	Mat. H	221	233	5.4	1.07	1.13
	Mat. G	215	231	7.4	1.04	1.12
Roof	Mat. H	107	112	4.7	1.03	1.08
	Mat. G	103	111	7.8	0.99	1.07

Table 5.16: Mean predicted maximum moment at the face of column with static and strain rate dependent material properties for limited ductile MRF (LD1)

Floor level	Material	Mean predicted maximum moment (kN.m)			$\frac{M_s}{M_{cf}}$	$\frac{M_r}{M_{cf}}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	666	715	7.4	1.05	1.13
	Mat. G	658	705	7.1	1.04	1.11
Second floor	Mat. H	669	718	7.3	1.05	1.13
	Mat. G	661	710	7.4	1.04	1.12
Third floor	Mat. H	238	255	7.1	1.07	1.14
	Mat. G	231	250	8.2	1.04	1.12
Roof	Mat. H	114	122	7.0	1.02	1.09
	Mat. G	111	121	9.0	0.99	1.08

Table 5.17: Mean predicted maximum moment at the centerline of column with static and strain rate dependent material properties for limited ductility MRF (LD1)

Floor level	Material	Predicted mean maximum moment (kN.m)			$\frac{M_s}{M_c}$	$\frac{M_r}{M_c}$
		Static, M_s	Rate, M_r	% increase over static		
First floor	Mat. H	739	795	7.6	1.05	1.13
	Mat. G	725	782	7.9	1.03	1.12
Second floor	Mat. H	745	801	7.5	1.06	1.14
	Mat. G	730	789	8.1	1.04	1.13
Third floor	Mat. H	268	287	7.1	1.07	1.14
	Mat. G	258	280	8.5	1.03	1.12
Roof	Mat. H	133.5	143	7.1	1.03	1.10
	Mat. G	129	140.5	8.9	0.99	1.08

Table 5.18: Predicted maximum inter-storey drift at first floor with static and strain rate dependent material properties for moderately ductile MRF (MD1), design 2.52%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.20	2.16	-1.9	2.20	2.16	-1.9
2	2.56	2.54	-0.8	2.54	2.52	-0.8
3	2.70	2.64	-2.3	2.70	2.65	-1.9
4	2.16	2.21	2.3	2.17	2.22	2.2
5	2.50	2.71	7.8	2.50	2.73	8.4
6	2.05	2.10	2.4	2.06	2.10	1.9
7	2.53	2.60	2.7	2.51	2.58	2.7
8	2.10	2.05	-2.4	2.10	2.06	-1.9
9	2.36	2.30	-2.6	2.36	2.31	-2.2
10	2.01	1.90	-5.8	2.04	1.87	-7.0
11	1.83	1.75	-4.6	2.02	1.76	-5.1
Mean	2.29	2.26	-0.5	2.27	2.27	-0.5

Table 5.19: Predicted maximum inter-storey drift at second floor with static and strain rate dependent material properties for moderately ductile MRF (MD1), design 2.39%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	1.98	1.95	-1.5	1.98	1.95	-1.5
2	2.16	2.12	-1.9	2.16	2.11	-2.3
3	2.20	2.27	3.2	2.20	2.28	3.6
4	1.85	1.87	1.1	1.85	1.87	1.1
5	2.15	2.16	0.5	2.14	2.15	0.5
6	1.95	1.93	-1.0	1.95	1.93	-1.0
7	2.10	1.98	-5.7	2.10	1.99	-5.2
8	1.52	1.50	-1.3	1.52	1.51	-0.7
9	1.95	1.91	-2.0	1.95	1.90	-2.6
10	1.75	1.75	0.0	1.75	1.74	-0.6
11	1.35	1.35	0.0	1.35	1.35	0.0
Mean	1.91	1.89	-0.8	1.90	1.89	-0.8

Table 5.20: Predicted maximum inter-storey drift at third floor with static and strain rate dependent material properties for moderately ductile MRF (MD1), design 2.45%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.35	2.33	-0.9	2.34	2.33	-0.7
2	2.23	2.21	-0.9	2.22	2.20	0.9
3	2.57	2.63	2.3	2.57	2.63	2.3
4	2.28	2.30	0.9	2.29	2.31	0.9
5	2.23	2.25	0.9	2.24	2.26	0.9
6	2.30	2.31	0.4	2.31	2.33	0.9
7	2.53	2.50	-1.2	2.51	2.48	-1.2
8	2.21	2.20	-0.4	2.24	2.23	-0.4
9	2.35	2.32	-1.3	2.35	2.32	-1.3
10	2.20	2.16	-1.9	2.19	2.15	-1.9
11	1.35	1.35	0.0	1.34	1.35	0.7
Mean	2.24	2.23	-0.2	2.24	2.23	-0.1

Table 5.21: Predicted maximum inter-storey drift at roof with static and strain rate dependent material properties for moderately ductile MRF (MD1), design 2.53%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.42	2.42	0.0	2.42	2.42	0.0
2	2.32	2.35	1.3	2.31	2.34	1.3
3	2.73	2.76	1.1	2.73	2.76	1.1
4	2.47	2.50	1.2	2.49	2.51	0.8
5	2.29	2.30	0.4	2.30	2.32	0.9
6	2.46	2.42	-1.7	2.45	2.42	-1.2
7	2.65	2.64	-0.4	2.62	2.61	-0.4
8	2.33	2.33	0.0	2.34	2.34	0.0
9	2.47	2.45	-0.8	2.48	2.46	-0.8
10	2.21	2.20	-0.4	2.20	2.18	-0.9
11	1.61	1.60	-0.6	1.61	1.60	-0.6
Mean	2.36	2.36	0.0	2.36	2.36	0.0

Table 5.22: Predicted maximum inter-storey drift at first floor with static and strain rate dependent material properties for moderately ductile MRF (MD2), design 3.01%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.80	2.77	-1.1	2.80	2.77	-1.1
2	3.05	3.01	-1.3	3.00	3.00	0.0
3	3.25	3.20	-1.5	3.20	3.20	0.0
4	2.70	2.52	-6.7	2.65	2.68	1.1
5	3.00	3.10	3.3	3.00	3.10	3.3
6	2.35	2.37	0.9	2.36	2.39	1.3
7	3.05	3.07	0.7	3.01	3.03	0.7
8	2.74	2.80	2.2	2.75	2.74	-0.4
9	2.70	2.65	-1.9	2.70	2.65	-1.9
10	2.30	2.29	-0.4	2.30	2.29	-0.4
11	2.40	2.31	-3.7	2.41	2.35	-2.5
Mean	2.76	2.74	-0.9	2.74	2.73	-0.6

Table 5.23: Predicted maximum inter-storey drift at second floor with static and strain rate dependent material properties for moderately ductile MRF (MD2), design 2.98%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.56	2.50	-2.3	2.56	2.50	-2.3
2	2.95	2.92	-1.0	2.94	2.92	-0.7
3	3.01	3.03	0.7	3.02	3.04	0.7
4	2.52	2.51	-0.4	2.52	2.51	-0.4
5	2.76	2.77	0.4	2.78	2.78	0.0
6	2.22	2.22	0.0	2.25	2.26	0.4
7	2.88	2.80	-2.8	2.85	2.79	-2.1
8	1.98	1.99	0.5	1.98	1.99	0.5
9	2.36	2.33	-1.3	2.35	2.33	-0.9
10	2.10	2.10	0.0	2.10	2.00	-4.8
11	1.85	1.85	0.0	1.85	1.85	0.0
Mean	2.47	2.46	-0.6	2.47	2.45	-0.9

Table 5.24: Predicted maximum inter-storey drift at third floor with static and strain rate dependent material properties for moderately ductile MRF (MD2), design 2.78%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.45	2.44	-0.4	2.44	2.44	0.0
2	2.75	2.69	-2.2	2.73	2.65	-2.9
3	2.77	2.79	0.7	2.77	2.79	0.7
4	2.23	2.23	0.0	2.24	2.23	-0.4
5	2.45	2.45	0.0	2.43	2.44	0.4
6	2.10	2.11	0.5	2.08	2.07	-0.5
7	2.48	2.45	-1.2	2.45	2.44	-0.4
8	1.75	1.75	0.0	1.74	1.73	-0.6
9	2.23	2.22	-0.4	2.20	2.20	0.0
10	2.00	2.00	0.0	1.98	1.98	0.0
11	1.78	1.79	0.6	1.77	1.77	0.0
Mean	2.27	2.27	-0.3	2.26	2.25	-0.4

Table 5.25: Predicted maximum inter-storey drift at roof with static and strain rate dependent material properties for moderately ductile MRF (MD2), design 2.80%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.49	2.49	0.0	2.49	2.49	0.0
2	2.83	2.85	0.7	2.84	2.85	0.4
3	2.79	2.80	0.4	2.78	2.79	0.4
4	2.51	2.48	-1.2	2.51	2.48	-1.2
5	2.56	2.56	0.0	2.56	2.56	0.0
6	2.23	2.21	-0.9	2.23	2.21	-0.9
7	2.56	2.56	0.0	2.56	2.56	0.0
8	1.98	1.95	-1.5	1.99	1.95	-2.0
9	2.36	2.35	-0.4	2.37	2.35	-0.8
10	2.20	2.20	0.0	2.19	2.18	-0.5
11	1.88	1.89	0.5	1.88	1.89	0.5
Mean	2.40	2.39	-0.2	2.40	2.39	-0.3

Table 5.26: Predicted maximum inter-storey drift at first floor with static and strain rate dependent material properties for limited ductility MRF (LD1), design 2.25%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.01	1.98	-1.5	2.01	1.98	-1.5
2	2.35	2.34	-0.4	2.35	2.34	-0.4
3	2.38	2.35	-1.3	2.38	2.35	-1.3
4	1.95	2.01	3.0	1.94	2.00	3.1
5	2.16	2.21	2.3	2.16	2.23	3.2
6	1.74	1.80	3.4	1.72	1.78	3.5
7	2.15	2.19	1.9	2.12	2.17	2.4
8	1.65	1.60	-3.0	1.65	1.60	-3.0
9	1.71	1.68	-1.8	1.77	1.68	-5.1
10	1.33	1.23	-7.5	1.33	1.22	-8.3
11	1.19	1.13	-5.0	1.19	1.12	-5.9
Mean	1.87	1.87	-0.9	1.87	1.86	-1.2

Table 5.27: Predicted maximum inter-storey drift at second floor with static and strain rate dependent material properties for limited ductility MRF (LD1), design 2.01%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	1.72	1.68	-2.3	1.72	1.68	-2.3
2	1.75	1.71	-2.3	1.74	1.70	-2.3
3	1.69	1.75	3.6	1.65	1.72	4.2
4	1.15	1.25	8.7	1.15	1.25	8.7
5	1.80	1.83	1.7	1.76	1.79	1.7
6	1.18	1.15	-2.5	1.15	1.12	-2.6
7	1.80	1.74	-3.3	1.70	1.69	-0.6
8	1.25	1.22	-2.4	1.25	1.22	-2.4
9	1.76	1.73	-1.7	1.76	1.72	-1.7
10	1.11	1.11	0.0	1.11	1.11	0.0
11	1.10	1.10	0.0	1.10	1.10	0.0
Mean	1.48	1.48	-0.4	1.46	1.46	0.7

Table 5.28: Predicted maximum inter-storey drift at third floor with static and strain rate dependent material properties for limited ductility MRF (LD1), design 2.23%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.25	2.22	-1.3	2.25	2.22	-1.3
2	2.15	2.13	-0.9	2.15	2.13	-0.9
3	2.35	2.39	1.7	2.35	2.39	1.7
4	2.15	2.17	0.9	2.17	2.18	0.5
5	2.19	2.20	0.5	2.20	2.24	1.8
6	2.21	2.24	1.4	2.22	2.25	1.3
7	2.45	2.43	-0.8	2.45	2.44	-0.4
8	2.19	2.17	-0.9	2.19	2.17	-0.9
9	2.25	2.21	-1.8	2.25	2.25	0.0
10	1.34	1.34	0.0	1.34	1.35	0.8
11	1.19	1.19	0.0	1.19	1.19	0.0
Mean	2.07	2.07	-0.1	2.07	2.07	0.2

Table 5.29: Predicted maximum inter-storey drift at roof with static and strain rate dependent material properties for limited ductility MRF (LD1), design 2.26%

ER	Inter-storey drift					
	Material H			Material G		
	%, Static	%, Rate	% change over static	%, Static	%, Rate	% change over static
1	2.27	2.27	0.0	2.27	2.27	0.0
2	2.22	2.24	0.9	2.22	2.24	0.9
3	2.39	2.41	0.8	2.39	2.42	1.3
4	2.22	2.23	0.4	2.23	2.23	0.0
5	2.25	2.27	0.9	2.24	2.26	0.9
6	2.22	2.21	-0.4	2.23	2.24	0.4
7	2.50	2.71	0.4	2.49	2.47	-0.8
8	2.22	2.21	-0.4	2.21	2.20	-0.4
9	2.33	2.34	0.4	2.34	2.35	0.4
10	1.39	1.37	-1.4	1.39	1.39	0.0
11	1.21	1.21	0.0	1.21	1.21	0.0
Mean	2.11	2.12	0.1	2.11	2.12	0.2

Table 5.30: Mean predicted inter-storey drift with static and strain rate dependent material properties at each floor level for moderately ductile MRF (MD1)

Floor level	Material	Inter-storey drift				
		%, Static	%, Rate	% change rate over static	Design (%)	% change rate over design
First floor	Mat. H	2.29	2.26	-0.5	2.52	-10.3
	Mat. G	2.27	2.27	-0.5		-9.9
Second floor	Mat. H	1.91	1.89	-0.8	2.39	-20.9
	Mat. G	1.90	1.89	-0.8		-20.9
Third floor	Mat. H	2.24	2.23	-0.2	2.45	-9.1
	Mat. G	2.24	2.23	-0.1		-9.1
Roof	Mat. H	2.36	2.36	0.0	2.53	-6.8
	Mat. G	2.36	2.36	0.0		-6.8

Table 5.31: Mean predicted inter-storey drift with static and strain rate dependent material properties at each floor level for moderately ductile MRF (MD2)

Floor level	Material	Inter-storey drift				
		%, Static	%, Rate	% change rate over static	Design (%)	% change rate over design
First floor	Mat. H	2.76	2.74	-0.7	3.01	-9.0
	Mat. G	2.74	2.73	-0.6		-9.3
Second floor	Mat. H	2.47	2.46	-0.6	2.98	-17.4
	Mat. G	2.47	2.45	-0.9		-17.8
Third floor	Mat. H	2.27	2.27	-0.3	2.78	-18.3
	Mat. G	2.26	2.25	-0.4		-19.1
Roof	Mat. H	2.40	2.39	-0.2	2.80	-14.6
	Mat. G	2.40	2.39	-0.3		-14.6

Table 5.32: Mean predicted inter-storey drift with static and strain rate dependent material properties at each floor level for limited ductility MRF (LD1)

Floor level	Material	Inter-storey drift				
		%, Static	%, Rate	% change rate over static	Design (%)	% change rate over design
First floor	Mat. H	1.87	1.87	-0.9	2.25	-16.9
	Mat. G	1.87	1.86	-1.2		-17.3
Second floor	Mat. H	1.48	1.48	0.4	2.01	-26.4
	Mat. G	1.46	1.46	0.7		-27.4
Third floor	Mat. H	2.07	2.07	-0.1	2.23	-7.2
	Mat. G	2.07	2.07	0.2		-7.2
Roof	Mat. H	2.11	2.12	0.1	2.26	-6.2
	Mat. G	2.11	2.12	0.2		-6.2

Table 5.33: Design and probable bending moment at the center of RBS for moderately ductile MRFs (MD1 and MD2), and limited ductility MRF (LD1).

Floor level	Moment (kN.m)		
	Design	Probable	Probable/ design
Moderately ductile MRF (MD1)			
First floor	387	520	1.34
Second floor	385	520	1.35
Third floor	135	184	1.36
Roof	69	96	1.39
Mean	244	330	1.36
Moderately ductile MRF (MD2)			
First floor	341	460	1.35
Second floor	334	449	1.34
Third floor	134	181	1.35
Roof	63	85	1.35
Mean	218	294	1.35
Limited ductility MRF (LD1)			
First floor	482	590	1.22
Second floor	482	590	1.22
Third floor	168	207	1.23
Roof	84	104	1.24
Mean	304	373	1.23

Table 5.34: Mean of predicted maximum base shear with static and strain rate dependent material properties compared to design base shear (V_d) 321 kN for MD1, 275 kN for MD2 and 467 kN for LD1

Material Properties	Predicted mean maximum base shear (kN)			$\frac{V_s}{V_d}$	$\frac{V_r}{V_d}$
	Static, V_s	Rate, V_r	% increase		
Moderately ductile MRF (MD1)					
Material H	365	376	3.0	1.14	1.17
Material G	349	356	2.0	1.09	1.11
Moderately ductile MRF (MD2)					
Material H	311	321	3.2	1.13	1.17
Material G	298	305	2.3	1.08	1.11
Limited ductility MRF (LD1)					
Material H	535	560	4.7	1.14	1.20
Material G	521	539	3.5	1.12	1.15

Table 5.35: Predicted mean strain hardening factors for moderately ductile MRF (MD1 and MD2) and limited ductility MRF (LD1) at the center of RBS with strain rate independent (static) material properties

Floor Levels	Moderately ductile MRF (MD1)		Moderately ductile MRF (MD2)		Limited ductility MRF (LD1)	
	Material H	Material G	Material H	Material G	Material H	Material G
First Level	1.19	1.25	1.17	1.26	1.09	1.17
Second Level	1.20	1.26	1.16	1.24	1.10	1.17
Third Level	1.20	1.25	1.19	1.26	1.07	1.13
Roof Level	1.20	1.25	1.15	1.23	1.06	1.11

Table 5.36: Predicted mean strain hardening factors for moderately ductile MRF (MD1 and MD2) and limited ductility MRF (LD1) at the center of RBS with strain rate dependent material properties

Floor Levels	Moderately ductile MRF (MD1)		Moderately ductile MRF (MD2)		Limited ductility MRF (LD1)	
	Material H	Material G	Material H	Material G	Material H	Material G
First Level	1.27	1.36	1.27	1.37	1.16	1.25
Second Level	1.28	1.37	1.25	1.35	1.16	1.26
Third Level	1.28	1.37	1.28	1.38	1.13	1.22
Roof Level	1.26	1.36	1.25	1.34	1.11	1.19

Figure 5.1: Matrix of parametric combinations considered in non-linear dynamic analyses

Figure 5.2: Target spectrum for scaling of earthquake records (NBCC 2015)

Figure 5.3: (a) Target spectra according to Method A (b) Scaling of ground motion records according to Method A (Tremblay et al. 2015)

Figure 5.4: (a) Target spectra according to Method B2 (b) Scaling of ground motion records according to Method B (Tremblay et al. 2015)

Figure 5.5: Response spectra of earthquake record suite for moderately ductile MRF (MD1)

Figure 5.6: Individually scaled earthquake record suite for moderately ductile MRF (MD1)

Figure 5.7: Scaled earthquake record suite for moderately ductile MRF (MD1)

Figure 5.8: Predicted maximum moment at the first floor of RBS for moderately ductile MRF (MD1) with strain rate dependent and independent (static) material properties

Figure 5.9: Predicted maximum moment at the roof of RBS for moderately ductile MRF (MD1) with strain rate dependent and independent (static) material properties

Figure 5.10: Predicted maximum moment at the first floor of RBS for moderately ductile MRF (MD2) with strain rate dependent and independent (static) material properties

Figure 5.11: Predicted maximum moment at the roof of RBS for moderately ductile MRF (MD2) with strain rate dependent and independent (static) material properties

Figure 5.12: Predicted maximum moment at the first floor of RBS for limited ductility MRF (LD1) with strain rate dependent and independent (static) material properties

Figure 5.13: Predicted maximum moment at the roof of RBS for limited ductility MRF (LD1) with strain rate dependent and independent (static) material properties

Figure 5.14: Maximum Inter-storey drift for moderately ductile MRF (MD1)

Figure 5.15: Maximum Inter-storey drift for moderately ductile MRF (MD2)

Figure 5.16: Maximum Inter-storey drift for limited ductility MRF (LD1)

Chapter 6: Conclusions and recommendations

6.1 Conclusions

The following conclusions can be drawn from the study.

1. Two finite element models of a beam-column connection with RBS using a combined shell and beam elements model (hybrid model), and a beam element only model have been explored by comparing against test results obtained from the experiments conducted by Chi and Uang (2002). The isotropic yield stress of yield stress versus true strain curves for materials H and G are scaled by a constant factor such that they match the initial yield strength of the material used in the experiment by Chi and Uang (2002). Good agreement has been achieved up to rotation of 0.02 between the test moment at the center of RBS versus rotation for specimen DC-3 and that of the predicted moment from the analyses with the hybrid model. However, beyond the rotation of 0.02, the results diverge as lateral torsional buckling was observed to have occurred in the test. This validates the suitability of the material properties and plasticity model that are used in the non-linear dynamic analyses. The beam element only model overpredicts the test moment, but the difference decreases with the increase in rotation. Due to the small differences in the results between the hybrid and beam element only models at high plastic deformation and a much higher computational efficiency of the beam element only model, the beam element only model was deemed acceptable to be used in the non-linear dynamic analyses.
2. In the experiments conducted by Chi and Uang (2002), the web and flange of the specimens have different yield strength. Numerical simulations using the hybrid model

with different combinations of web and flange material properties showed that the whole beam can be modelled as a single material using the material properties of the flange.

3. In the preliminary study, two single storey moderately ductile moment resisting frames with 32% and 46% maximum flange width reduction for the RBS have been modelled. Results of the non-linear dynamic analyses of the preliminary models suggested that the frame with 46% of maximum flange width reduction has a higher rate of increase in the moment at the center of RBS due to strain rate dependency of material properties. Hence, all four-storey moment resisting frames have been designed with the maximum flange width cut ratio close to 50% in order to obtain maximum effects of material properties strain rate dependency from the analyses.
4. The maximum strain rate produced at the center of RBS for MD1 and MD2 is 0.18s^{-1} , and for LD1 is 0.19s^{-1} . An increase in the maximum strain rate with storey height and, a lack of dependency of the strain rate on material properties and design ductility level of MRF with RBS has been observed. Nevertheless, the strain rate is sufficiently high that effects of material properties strain rate dependency on other aspects of the structural response can be important.
5. The mean predicted inter-storey drift with strain rate dependent material properties vary 6.2% to 27.4% below the design inter-storey drift. The maximum difference in the mean predicted inter-storey drift between materials H and G is 0.02%, and the maximum difference between with and without strain rate dependency in material properties is 0.03%. The negligible difference in the results shows that the strain rate dependency in material properties and different material strength do not have significant effect on the mean predicted inter-storey drift for the MRFs considered.

6. The strain rate dependency of material properties has been found to greatly affect the moment generated in the beam at the center of RBS. Analyses have shown that the mean (for a suite of ground motions) increase in the moment can be as high as 10% with the consideration of material properties strain rate dependency when the inter-storey drift limit is not a design constraint, and up to 9% when the MRF is designed to meet the inter-storey drift limit. The increase is also higher for the moderately ductile MRF than the limited ductility MRF. Even without the consideration of material properties strain rate dependency, the mean predicted maximum moment at the center of RBS with material H exceeds the calculated probable moment. With the consideration of material properties strain rate dependency, the mean predicted maximum moment can be higher than the calculated probable moment at the center of RBS by up to 13%.
7. There is an increase in the mean predicted maximum base shear with strain rate dependent material properties. However, the increase is only about half that of the moment at the center of RBS. Also unlike the moment at the center of RBS, the increase is higher for material H than for material G even though material G has a higher isotropic yield stress amplification factor with strain rate. Overall, the mean predicted maximum base shear exceeds the design base shear close to 14% for material H with static material properties, and 20% with strain rate dependent material properties.
8. Similar to the factor applied in the design provision, the predicted strain hardening factor from the analyses for moderately ductile MRFs (MD1 and MD2) is about 10% higher than the factor for limited ductility MRF (LD1). The predicted mean strain hardening factor is higher by up to 15% over the prescribed factors in the design. With strain rate

dependent material properties, the predicted mean strain hardening factor calculated with respect to the initial static yield stress exceeds the prescribed factors by up to 25%.

6.2 Recommendations

1. The four-storey moment resisting frames designed in this study are two-dimensional models that are subjected to horizontal ground motions only in one direction (lateral direction). A multidirectional seismic analyses of a three-dimensional structure subjected to a pair of orthogonal horizontal ground motion records can be analysed in order to provide a more comprehensive study of the building response.
2. The moment resisting frames are subjected to a suite of ground motion records which are scaled according to Method A in NBCC 2015 in the current study. Analyses with earthquake ground motions scaled with Methods B1 and B2 can be considered to assess the effects of material properties strain rate dependency with different scaling procedures.
3. In this study, only four-storey moderately ductile and limited ductility MRFs are designed and analysed. Frames with different storeys can be considered for further studies.
4. The RBS has been designed with the ratio of the maximum flange width cut close to 50% in this study. A more comprehensive analyses can be carried out on frames to assess effects of material properties strain rate dependency for a range of cut ratios.
5. Shake table testing of moderately ductile and limited ductility moment resisting frames with reduced beam sections can be carried out to validate the analytical results obtained from this study.

REFERENCES

1. Abrahamson, N. A. (1992). "Non-stationary spectral matching." *Seismological Research Letters*, 63(1), 30.
2. AISC (1997). "Seismic Provisions for Structural Steel Buildings." American Institute of Steel Construction, Chicago, USA, edition 2.
3. Amirzehni, E., Taiebat, M., Finn, W. L., & DeVall, R. H. (2015). "Ground motion scaling/matching for nonlinear dynamic analysis of basement walls."
4. ANSI/AISC 358-16 (2016). "Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications." American Institute of Steel Construction, USA, 2016.
5. Armstrong, P. J., and Fredrick, C. O. (1966). "A Mathematical Representation of the Multiaxial Bauschinger Effect." CAGB Report, RD/B/N/731, Berkeley Nuclear Laboratories, R&D Department, California, USA.
6. Applied Technology Council (1995). "Structural Response Modification Factors." ATC-19, National Science Foundation, Redwood City, California, USA.
7. Atkinson, G. M., & Adams, J. (2013). "Ground motion prediction equations for application to the 2015 Canadian national seismic hazard maps." *Canadian Journal of Civil Engineering*, 40(10), 988–998.
8. Chaboche, J. L., Dang Van, K., and Cordier, G. (1979). "Modelization of the Strain Memory Effect on the Cycle Hardening of 316 Stainless Steel, Transaction of IASMiRT 5." Vol. L, L11/3.
9. Chang, K. C., and Lee, G. C. (1987). "Strain Rate Effect on Structural Steel under Cyclic loading." *Journal of Engineering Mechanics* 113, no 9, 1292-1301.

10. Chen, J. (2010). “ An Experimental Study of Strain Rate Effects on Mild Steel.”
The Department of Civil Engineering, Carleton University, Ottawa, ON, Canada.
11. Chen, S. J., Yeh, C. H., and Chu, J. M. (1996). “Ductile steel beam to column connections for seismic resistance.” J. Struct. Eng., 122(11), 1292-1299.
12. Chi, B., and Uang, C. M. (2002). “Cyclic Response and Design Recommendations of Reduced Beam Section Moment Connections with Deep Columns.” Journal of Structural Engineering, 128(4), 464-473.
13. CISC. (2014). “Moment Connections for Seismic Applications.” Canadian Institute of Steel Construction, Markham, Ontario, Canada, edition 2.
14. Clark, P., Frank, K., Krawinkler, H., and Shaw, R. (1997). “Protocol for Fabrication, Inspection, Testing and Documentation of Beam-Column Connection Tests and Other Experimental Specimens.” SAC joint venture, Sacramento, California, USA.
15. Cowper, G. R., and Symonds, P. S. (1957). “Strain-Hardening and Strain-Rate effects in the Impact Loading of Cantilever Beams.” Division of Applied Mathematics, Brown University, Providence, R.I. USA.
16. Canadian Standards Association (CSA) (2014). “Design of Steel Structures.” CAN/CSA-S16-14, Mississauga, Ontario, Canada.
17. El-Tawil, S., and Deierlein G. G. (2001). “Nonlinear analysis of mixed steel-concrete moment frames Part 1- Beam-column element formulation.” Journal of Structural Engineering, ASCE.
18. Engelhardt, M. D., Winneberger, T., Zekany, A. J., and Potyraj, T. (1996). “The Dogbone Connection: Part II.” Modern Steel Construction, 36(8), 46-55.

19. Engelhardt, M. D. (1998). "Design of Reduced Beam Section Moment Connection." North American Steel Construction Conference, AISC, Chicago, 1-29.
20. Eyring, H. (1936). "Viscosity, plasticity and diffusion as examples of absolute reaction rates." The journal of Chemical Physics, vol. 4, no. 4, 283-291.
21. Federal Emergency Management Agency (FEMA) (2000). "Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings." FEMA-350. Washington DC, USA.
22. Federal Emergency Management Agency (FEMA) (2009). "Quantification of Building Seismic Performance Factors." FEMA-P695. Washington DC, USA.
23. Halchuk, S., Adams, J. E., & Allen, T. I. (2015). "Fifth generation seismic hazard model for Canada-grid values of mean hazard to be used with the 2015 National Building Code of Canada ." (No. 7893), Natural Resources Canada.
24. Hamburger R. O., and Nazir N. A. (2003). "Seismic Design of Steel Structures." Earthquake Engineering Handbook, CRC press, Florida, USA.
25. Iwankiv, N. R., and Carter, C. J. (1996). "The Dogbone: A New Idea to Chew On." Modern Steel Construction, 36(4), 18-23.
26. Jin, J., and El-Tawil, S. (2004). "Seismic Performance of Steel Frames with Reduced Beam Section Connections." Journal of Constructional Steel Research, 61, 453-471.
27. Jones, S. L., Fry, G. T., and Engelhardt, M. D. (2002). "Experimental Evaluation of Cyclically Loaded Reduced Beam Section Moment Connections." Journal of Structural Engineering, 128(4), 441-451.

28. Kaufmann, E. J., Fisher, J. W., Di Julio, R. M., and Gross, J. L. (1997). "Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake." Report NISTIR 5944, National Institute of Standards and Technology.
29. Krawinkler, H. (1978). "Shear in beam-column joints in seismic design of steel frame." Engineering journal, AISC 15(3), 82-91.
30. Krempl, E. (1979). "An Experimental Study of Room-Temperature Rate-Sensitivity, Creep and Relaxation of AISI Type 304 Stainless Steel." Journal of the Mechanics and Physics of Solids, 27(5-6), 363-375.
31. Kudu, F. N., Ucak, S., Osmancikli, G., Turker, T., and Bayraktar, A. (2015). "Estimation of Damping Ratios of Steel Structures by Operational Modal Analysis Method." Journal of Constructional Steel Research, volume 112, p 61-68.
32. Lee, C. H., and Kim, J. (2004). "Seismic Design of Reduced Beam Section Steel Moment Connections with Bolted Web Attachment." Journal of Constructional Steel Research, 63, 522-531.
33. Lemaitre, J., and Chaboche, J. L., (1990). "Mechanics of Solid Materials." Cambridge University Press, Cambridge.
34. Li, F. X., Kanao, I., Li, J., and Morisako, K. (2009). "Local buckling of RBS beams subjected to cyclic loading." J. Struct. Eng., 10.1061/ (ASCE)ST.1943-541X.0000073, 1491–1498.
35. Li, M., and Li, H. (2011). "Effects of Strain Rate on Reinforced Concrete Structure under Seismic Loading." Advances in Structural Engineering, vol. 15 (3).

36. Manjoine, M. J. (1944). "Influence of Rate of Strain on Temperature on Yield Stresses of Mild Steel." Transactions of the American Society of Mechanical Engineers, Journal of Applied Mechanics, 66: A211-A218.
37. Metten, A., and Driver, R. (2015). "Structural Steel for Canadian Buildings." Library and Archives Canada Cataloguing in Publication, Canada , edition 3.
38. NBCC (2015). "National Building Code of Canada." Canadian Code on Building and Fire Code, National Research Council, vol. 4, edition 15.
39. Othman, R. (2015). "A Modified Eyring Equation for Modelling Yield and Flow Stresses of Metals at Strain Rates Ranging from 10^{-5} to 5×10^4 s $^{-1}$." Advances in Materials Science and Engineering, vol 2015.
40. Orban, F. (2011). "Damping of Materials and Members in Structures.". Journal of Physics Conference Series. 268,012022.
41. Plumier, A. (1997). "The Dogbone: Back to the Future." Engineering Journal, AISC, 34(2), 61-67.
42. Popov, E. P., and Stephen, R. M. (1970). "Cyclic Loading of Full-Size Steel Connections." American Iron and Steel Institute Bulletin, 21.
43. Popov, E. P., Yang, T. S., and Chang, S. P., (1998). "Design of Steel MRF Connections Before and After 1994 Northridge Earthquake." Engineering Structures, 20(12), 1030-1038.
44. Prakash, V., Powell, G. H., and Campbell, S. (1993). "DRAIN-2DX Base Program Description and User Guide." version 1.10, University of California, Berkeley.
45. Roeder, C. (2000). "State of the Art Report on steel connections, FEMA-355D." Federal Emergency Management Agency, Washington D.C., USA.

46. Salahi, A. A. A, and Othman, R. (2016). "Constitutive Equations of Yield Stress Sensitivity to Strain Rate of Metals: A Comparative Study." *Journal of Engineering*, volume 2016.
47. Shen, J., Kitjasateanphun, T., and Srivanich, W. (1999). "Seismic Performance of Steel Moment Frames with Reduced Beam Sections." *Engineering Structures*, 22, 968-983.
48. Simulia (2017). Abaqus 6.17, Dassault Systems Simulia Corp., Providence, Rhode Island, USA.
49. Singh, T. (2019). "Strain Rate Effects on Ductile Moment Resisting Frame with RBS Connection Subjected to Seismic Ground Motion." Department of Civil and Environmental Engineering, Carleton University, Ottawa, Ontario, Canada.
50. Tian, L., Wang, W., and Qian, H. (2014). "The Effect Analysis of Strain Rate on Power Transmission Tower-Line System under Seismic Excitation." *The Scientific World Journal*, 2014.
51. Tremblay, R., Atkinson, G. M., Bouaanani, N., Daneshvar, P., Leger, P., and Koboevic, S. (2015). "Selection and scaling of ground motion time histories for seismic analysis using NBCC 2015." 11th Canadian Conference on Earthquake Engineering.
52. Uang, C. M., and Bertero, V. V. (1986). "Earthquake Simulation Tests and Associated Studies of a 0.3 scale Model of a Six-storey concentrically Braced Steel Structure". Report No. UCB/EERC-86/10. Berkeley. Earthquake Engineering Research Center. University of California.

53. University of Washington and University of Oregon (1969). "Pacific Northwest Seismic Network (PNSN)." U.S. Geological Survey (pnsn.org).
54. Wakabayashi, M., Nakamura, T., Iwai, S., & Hayashi, Y. (1984). "Effects of Strain Rate on the Behaviour of Structural Members Subjected to Earthquake Force". Proc. 8th World Conference on Earthquake Engineering, San Francisco, USA.
55. Walker, A. (2012). "An Experimental and Numerical Study of the Strain Rate Effects on Mild Steel for Cyclic Loading." The Department of Civil and Environmental Engineering, Carleton University, Ottawa, ON, Canada.
56. Wang, Y., Lin, D., Zhou, Y., Xia, Y., and Law, C. C. (1999), "Dynamic Tensile Properties of Ti-47Al-2Mn-2Nb alloy." Journal of Materials Science, vol. 34, no. 3, 509–513.
57. Yang, T. S., and Popov, E. P. (1995). "Behavior of Pre-Northridge Moment Resisting Steel Connections." UCB/EERC-95/08, Earthquake Engineering Research Center, University of California, Berkeley, USA.
58. Zekioglu, A., Mozaffarian, H., and Uang, C. M. (1997). "Moment Frame Connection Development and Testing for the City of Hope National Medical Center." Structure Congress XV, vol. 1, ASCE, N.Y., 100-104.
59. Zhang, H., and Li, H. N. (2011). "Dynamic Analysis of Reinforced Concrete Structure with Strain Rate Effect." Springer Berlin, 213-216.
60. Zhang, X., and Ricles, J. M. (2006). "Experimental Evaluation of Reduced Beam Section Connections to Deep Columns." Journal of Structural Engineering, 132(3), 346-357.

Appendix A: Design of moderately ductile and limited ductility moment resisting frames

In this appendix, equations and procedures for seismic design of a moment resisting frames are described. Structural members are designed according to provisions in CSA S16-14 (CSA 2014). Loads acting on these members such as dead load, live load, snow load and earthquake load are calculated according to National Building Code of Canada (NBCC 2015).

A four-storey building designed by Jin and El-Tawil (2004) is taken as a reference for the design of the four-storey office building in this study. It is a 4-bay square building with identical sides as shown in Fig. A.1. For simplicity, the span of the bay is rounded to 9 m and the height of each storey is rounded to 4 m. There are two moment resisting frames on each side of the perimeter with the moment and non-moment frames arranged alternately and MRFs on each side of the building are identified with “MF” in Fig. A.1. The office building is assumed to be located in Powell River, British Columbia on a class site D. Powell River, British Columbia is located in a high seismic hazard zone as shown in Fig. A.2. Both low ductile and moderately ductile moment resisting frames with RBS are considered for the office building with an importance factor of 1.0.

A 1. Load calculations

The snow load on the structure is calculated according to the National Building Code of Canada (NBCC 2015) Clause 4.1.6.2 as followed,

$$S = I_s * (S_r * (C_b * C_w * C_s * C_a) + S_r) \quad (A.1)$$

where I_s is the snow load importance factor of the building, S_r and S_s represent parameters associated with rain load and ground snow load respectively for a return period of 50 years, C_b is the basic roof snow load multiplying constant (usually 0.8 for roofs having less than 70m width and length respectively for a square building), C_s is the slope factor, C_w is the wind factor and C_a is the snow accumulation factor. Table A.1 shows the values of parameters in Eq. (A.1) used to calculate the snow load on the building.

For an office building, the live load is 1.0 kPa for the roof and 2.4 kPa for the other floors with a live load reduction factor of 0.35. The dead load on each level is taken to be the same as that of the building by Metten and Driver (2015). Table A.2 shows the breakdown of the dead load for the roof and other floors. Calculations of seismic weight and live load of the building are shown in Table A.3. Load combination considered is (1.0 D+1.0 E) as principal load and (0.5 L+ 0.25 S) as companion load. Here, only 50% of live load is considered. Taking the live load reduction factor of (0.35) for the total floor area of 3 storey, the live load acting on each floor is $0.5 \times 0.35 \times 2.4$, which is 0.42 kPa. Only 25% of the snow load at roof level contribute to the seismic weight, which is 25% of 1.92 kPa or 0.48 kPa. The roof and floor area on each level is 36 m x 36 m (1296 m²). Multiplying the area with seismic weights, the seismic load 2786 kN and 4303 kN are obtained for the roof and each of the floor respectively. Taking the tributary width of beam as 1m, the gravity load distribution on the frame is shown in Fig. A.3.

A 2. Base shear calculation using static procedure

The seismic forces on the building are calculated according to the static procedure as outlined in NBCC 2015 Clause 4.1.8. The initial fundamental time period of the structure is considered to be $0.085h^{3/4}$ where h is the total height of the building. It was calculated to be 0.68s in this case. The design spectrum of Powell River, British Columbia is shown in Fig. A.4 and the values are listed in Table A.4. According to NBCC 2015 Clause 4.1.8.1, the minimum lateral seismic force at the base of the structure can be calculated with

$$V = \frac{S(T_a)M_v I_E W}{R_d R_o} \quad (A.2)$$

where $S(T_a)$ is the spectral acceleration at the given time period T_a . The initial fundamental time period is 0.68s for moment resisting frame considered in the study. M_v (considered as 1.0) is the higher mode factor, I_E (1.0 for office buildings) is the importance factor of the building, W is the total seismic weight of the building, R_d is the ductility related to force modification factor and R_o is the overstrength related to force modification factor (3.5 and 1.5, and 2.0 and 1.3 respectively for moderately ductile MRF and limited ductility MRF). According to the time period, the total base shear was calculated to be 311 kN and 271 kN for moderately ductile MRFs (MD1 and MD2) respectively and 453 kN for limited ductility MRF (LD1).

According to NBCC 2015 Clause 4.1.8.7, the calculated base shear for moment resisting frames should satisfy the condition in Clause 4.1.8.11

$$V > \frac{S(2.0)M_v I_E W}{R_d R_o} \quad (A.3)$$

where $S(2.0)$ represents the spectral acceleration at time period 2s.

Moreover, for a building located on site class other than class F and having R_d

greater than or equal to 1.5, the calculated V should not be greater than the larger value obtained from the two equations

$$\frac{2}{3} \frac{S(a)I_E W}{R_d R_o} \text{ and} \quad (A.4)$$

$$\frac{S(0.5)I_E W}{R_d R_o} \quad (A.5)$$

The total lateral seismic force can be distributed over the height of the building according to NBCC 2015 Clause 4.1.8.11. If the fundamental time period of the building is greater than 0.7s, a concentrated force F_t should be applied at the roof level of the structure. For the building with time period less than 0.7s, F_t is considered as zero. The concentrated force is taken as

$$F_t = 0.07T_a V \quad (A.6)$$

where T_a is the fundamental lateral time period and V is the total base shear. The value of F_t should not exceed 0.25V. The remaining lateral seismic force is distributed along the height of the building including the top level (roof) of the building according to the equation

$$F_x = (V - F_t) * \frac{W_x * h_x}{\sum_i^n W_i h_i} \quad (A.7)$$

where F_x is the lateral force applied at level x, W_x and W_i are the seismic weight at respective floor levels x and i, and h_x and h_i are the height of the building from the base to the levels x and i. The building is considered to have two moment resisting frames on each side of the perimeter, and thus four frames per direction. Therefore, the force on each MRF will be a quarter of the total seismic shear force. The seismic force at each storey of

a single MRF is listed in Table A.5. Since there are eight moment resisting frames around the perimeter of the square building, the accidental torsion due to the seismic force applied at an eccentricity of 10% of the building dimension is

$$T_E = 0.1D F_x \text{ or } 0.1D (F_t + F_x) \quad (A.8)$$

where D is the width of the building. Thus, the force added on each frame can be calculated by

$$F_{xt} = \frac{T_E}{8 (0.5 D)} = 0.025 F_x \text{ or } 0.025 (F_t + F_x) \quad (A.9)$$

Notional loads due to building out of plumb are applied at 0.005 of the gravity load of each floor, and are divided for four frames are added to the seismic forces. The final loadings on the frames are shown in Figs. A.3 and A.5 to A.7.

A.3 Design of reduced beam section (RBS)

The design of reduced beam section (RBS) connections in the seismic force resisting steel moment frame is carried out according to the guidelines in Moment Connections for Seismic Applications (CISC 2014). As shown in Fig. A.8, the reduced beam section is designed by choosing parameters a, s and c such that

$$0.5b \leq a \leq 0.75b \quad (A.10)$$

$$0.65d_b \leq s \leq 0.85d_b \quad (A.11)$$

$$0.1b \leq c \leq 0.25b \quad (A.12)$$

where b is the beam flange width, d_b is the beam depth, a is horizontal distance from face of column, s is the length of RBS cut and c is the depth of cut at center of RBS. From these chosen dimensions, the plastic section modulus at the center of RBS (Z_e) can be calculated as

$$Z_e = Z_b - 2ct_b(d_b - t_b) \quad (A.13)$$

where Z_b is the plastic section modulus and t_b is the thickness of flange of the beam. From the value of Z_e , the maximum probable moment at the center of RBS (M_{pr}) can be computed as

$$M_{pr} = C_{pr}R_yF_yZ_e \quad (A.14)$$

where C_{pr} is the factor that take into consideration the plastic hinging effects such as strain hardening ($C_{pr}= 1.0$ for limited ductility MRF and $C_{pr}= 1.1$ for moderately ductile MRF), R_y is the ratio of probable yield stress to minimum yield stress ($R_y = 1.1$). The shear force at the center of RBS (V_{RBS}) can be calculated as

$$V_{RBS} = \frac{2M_{pr}}{L} \pm \frac{wL}{2} \quad (A.15)$$

where w is the uniformly distributed gravity load on the beam and L is the length between centers of RBS cuts. The moment generated at the face of the column (M_{cf}) due to the plastic hinge formation at center of RBS can be calculated as

$$M_{cf} = M_{pr} + V_{RBS} \left(a + \frac{s}{2} \right) \quad (A.16)$$

Similarly, the moment generated at the centerline of column due to the plastic hinge formation can be calculated as

$$M_c = M_{pr} + V_{RBS} \left(a + \frac{s}{2} + \frac{d_c}{2} \right) \quad (A.17)$$

where d_c is the depth of the column. The moment generated at the face of the column (M_{cf}) should be less than or equal to the plastic moment capacity of the beam. The plastic moment formation due to expected yield stress can be calculated from

$$M_{pe} = Z_bR_yF_y \quad \text{with} \quad (A.18)$$

$$\frac{M_{cf}}{M_{pe}} \leq 1.0 \quad (A.19)$$

For a beam to form a plastic hinge during the seismic excitation, the moment frame should be designed according to the strong column-weak beam concept. The beam members for the three frames are given in Tables A.6 to A.8. The dimensions of RBS cut are provided in Tables A.9 to A.11 (refer to Fig. A.8 for details of RBS) for the beam at each levels along with the design summaries of RBS in Tables A.12 to A.14. Overall, the RBS cut for all the three MRFs ranges from 45% to 50% reduction of the beam width.

The column should be designed such that the moment generated during the plastic hinge formation does not result in its failure. The column is designed as beam column according to CSA S16-14 Clause 13.8.2 (CSA 2014)

$$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x} \leq 1.0 \quad (A.20)$$

where C_r is the factored compressive resistance, M_x is the maximum factored moment resistance, C_f is the axial force on the column calculated according to the Clause 27.2.3.2, M_f is the moment generated at the column centerline and U_1 is the factor for the second-order effect due to axial force. The value of C_r is calculated according to Clause 13.3.1 in CSA S16-14 (CSA 2014) as

$$C_r = \frac{\phi A F_y}{(1 + \lambda^n)^{\frac{1}{n}}} \quad (A.21)$$

where ϕ is the resistance factor, A is the area, λ is non-dimensional slenderness parameter and n is 1.34. As can be seen in Tables A.15 to A.17 for the design of column, values of Eq. (A.20) for MRF MD2 are closer to 1.0, and are higher compared to those for MRFs MD1 and LD1. Values for MRFs MD1 and LD1 are lower because these two frames are

designed to meet the inter-storey drift limit. The ratio beside M_f in Tables A.15 to A.17 represents the ratio of the beam moment to column moment at the joint of first floor based on static analysis. In the design, the moment M_c of the beam from Eq. (A.17) is used as the column moment M_f , and is not distributed to the upper and lower columns at the joint. The calculated inter-storey drift of the frames is listed in Table A.18. As can be seen in the table, for the inter-storey drifts are very close to (only 1% larger) or less than 2.5% for MRFs MD1 and LD1. However, for MRF MD2, the inter-storey drifts are higher than the 2.5% limit. The drift is as high as 3.01 because MD2 is designed entirely for strength alone and not having to meet the inter-storey drift limit.

The initial fundamental time period according to the equation $0.085 h^{3/4}$ has been calculated to be 0.68s. Upon further iterations of the design, time periods for the final design are 1.45s and 1.75s for moderately ductile MRFs MD1 and MD2 respectively, and 1.24s for limited ductility MRF LD1. The total base shear for MRF MD1, MD2 and LD1 was calculated to be 321 kN, 275 kN and 467 kN respectively, and its contribution on each storey has been given in Table A.5. The seismic shear forces, seismic forces generated by accidental torsion and notional loads on a single MRF are shown in Figs. A.5 and A.6 for moderately ductile MRFs MD1 and MD2, and in Fig. A.7 for the limited ductility MRF LD1. Member sizes for MRFs MD1, MD2 and LD1 are shown in Fig. A.9.

Table A.1: Snow load parameters

Importance factor (I_s)	1.0
Load associated to ground snow (S_s), (kPa)	1.9
Load associated to rain (S_r), (kPa)	0.4
Basic roof snow load constant (C_b)	0.8
Slope factor (C_s)	1.0
Wind factor (C_w)	1.0
Snow accumulation factor (C_a)	1.0
Total snow load (kPa)	1.92

Table A.2: Dead load on each level of the building (Metten and Driver 2015)

Roof (kPa)		First floor/second floor/third floor (kPa)	
Roofing/ deck/ insulation	1.00	Partitions	0.50
Mechanical/ electrical	0.10	65 toppings on 38 deck	2.40
Steel beams and joists	0.32	Mechanical/ electrical	0.10
Cladding/ partition	0.25	Floor joists	0.22
		Girders and tributary column	0.10
Total dead load at roof level (kPa)	1.67	Total dead load at floor level (kPa)	3.32

Table A.3: Total seismic weight and live load on each level

	Roof	First floor/second floor/third floor
Area	36 m x 36 m	36 m x 36 m
Snow load (kPa) (25% of total snow load)	0.48	-
Dead load (kPa)	1.67	3.32
Total dead + snow load (kPa)	2.15	3.32
Total seismic weight (kN)	2786	4302
Live load (kPa) 50%	-	0.42
Total live load (kN)	-	544

Table A.4: Design spectrum for Powell River, British Columbia for site class D

Time period (T)	Spectral acceleration $S_a(T)$
0.0	0.682
0.2	0.682
0.5	0.682
1.0	0.497
2.0	0.334
5.0	0.123
10.0	0.043

Table A.5: Seismic shear force distribution at each storey of moment resisting frame

Level	Area (m ²)	W _i /Area a	Weight (W _i)	Height H _i (m)	W _i H _i	W _i H _i / $\sum W_i H_i$	Shear force, V (kN)		
							MD 1	MD 2	LD 1
Roof	1296	2.15	2786	16	44576	0.30	97	83	140
Third	1296	3.32	4303	12	51636	0.35	112	96	162
Second	1296	3.32	4303	8	34424	0.23	75	64	109
First	1296	3.32	4303	4	17212	0.17	37	32	56
		Sum	15695		147848		321	275	467

Table A.6: Sectional properties of structural members for moderately ductile MRF MD1

	Column W840x176	First floor beam W530x74	Second floor beam W530x74	Third floor beam W360x39	Roof beam W310x24
Depth of cross-section d (mm)	835	529	529	353	305
Width of flanges b (mm)	292	166	166	128	101
Thickness of flanges t (mm)	18.8	11.4	11.4	10.7	6.7
Thickness of web w (mm)	14.0	8.9	8.9	6.5	5.6
Moment of inertia I_x (mm ⁴)	2460x10 ⁶	411x10 ⁶	411x10 ⁶	102x10 ⁶	42.7X10 ⁶
Section modulus Z_x (mm ³)	6810x10 ³	1810x10 ³	1810x10 ³	662 x10 ³	328X10 ³
Design moment on the beam at face of the column M_f (kN.m)	-	570	562	200	95

Table A.7: Sectional properties of structural members for moderately ductile MRF MD2

	Column W610x101	First floor beam W460x74	Second floor beam W460x74	Third floor beam W310x45	Roof beam W250x25
Depth of cross-section d (mm)	603	457	457	313	257
Width of flanges b (mm)	228	190	190	166	102
Thickness of flanges t (mm)	14.9	14.5	14.5	11.2	8.4
Thickness of web w (mm)	10.5	9.0	9.0	6.6	6.1
Moment of inertia I_x (mm ⁴)	764x10 ⁶	333x10 ⁶	333x10 ⁶	99.2x10 ⁶	34.2 x10 ⁶
Section modulus Z_x (mm ³)	2900x10 ³	1650x10 ³	1650x10 ³	708x10 ³	307x10 ³
Design moment on the beam at face of the column M_f (kN.m)	-	499	485	197	96

Table A.8: Sectional properties of structural members for limited ductility MRF LD1

	Column W840x193	First floor beam W610x82	Second floor beam W610x82	Third floor beam W360x51	Roof beam W310x28
Depth of cross-section d (mm)	840	599	599	355	309
Width of flanges b (mm)	292	178	178	171	102
Thickness of flanges t (mm)	21.7	12.8	12.8	11.6	8.9
Thickness of web w (mm)	14.7	10.0	10.0	7.2	6.0
Moment of inertia I_x (mm ⁴)	2780x10 ⁶	565x10 ⁶	565x10 ⁶	141x10 ⁶	54.3 x10 ⁶
Section modulus Z_x (mm ³)	7620x10 ³	2210x10 ³	2210x10 ³	894x10 ³	407x10 ³
Design moment on the beam at face of the column M_f (kN.m)	-	657	648	235	118

Table A.9: Dimensions of RBS for moderately ductile MRF MD1

Floor level	Transition zone a, mm	Length of RBS s, mm	Depth of RBS cut c, mm
First floor beam	120	450	41.5
Second floor beam	120	450	41.5
Third floor beam	65	230	30.9
Roof beam	65	250	25.2

Table A.10: Dimensions of RBS for moderately ductile MRF MD2

Floor level	Transition zone a, mm	Length of RBS s, mm	Depth of RBS cut c, mm
First floor beam	140	380	44.0
Second floor beam	140	380	46.0
Third floor beam	120	260	41.5
Roof beam	75	200	25.2

Table A.11: Dimensions of RBS for limited ductility MRF LD1

Floor level	Transition zone a, mm	Length of RBS s, mm	Depth of RBS cut c, mm
First floor beam	90	400	44.5
Second floor beam	90	400	44.5
Third floor beam	90	300	42.5
Roof beam	75	250	25.5

Table A.12: Design summary of RBS for moderately ductile MRF MD1

	First floor beam	Second floor beam	Third floor beam	Roof beam
Plastic section modulus at the center of RBS Z_e (mm ³)	1.23x10 ⁶	1.23x10 ⁶	4.35x10 ⁵	2.27x10 ⁵
Probable maximum moment at the center of RBS M_{pr} (kN.m)	520	520	184	96
Maximum moment at the face of the column M_{cf} (kN.m)	569	569	195	104
Maximum moment at the centerline of column M_c (kN.m)	628	628	221	120
V_h (kN)	142	142	60	39
$\frac{M_{cf}}{M_{pe}}$	0.82	0.82	0.77	0.82
Design moment at the center of RBS M_f (kN.m)	387	385	135	69

Table A.13: Design summary of RBS for moderately ductile MRF MD2

	First floor beam	Second floor beam	Third floor beam	Roof beam
Plastic section modulus at the center of RBS Z_e (mm ³)	1.08x10 ⁶	1.06x10 ⁶	4.27x10 ⁵	2.02x10 ⁵
Probable maximum moment at the center of RBS M_{pr} (kN.m)	460	449	181	85
Maximum moment at the face of the column M_{cf} (kN.m)	501	489	196	92
Maximum moment at the centerline of column M_c (kN.m)	540	528	214	103
V_h (kN)	127	124	60	37
$\frac{M_{cf}}{M_{pe}}$	0.79	0.77	0.72	0.77
Design moment at the center of RBS M_f (kN.m)	341	334	134	63

Table A.14: Design summary of RBS for limited ductility MRF LD1

	First floor beam	Second floor beam	Third floor beam	Roof beam
Plastic section modulus at the center of RBS Z_e (mm ³)	1.53x 10 ⁶	1.53x 10 ⁶	5.54 x 10 ⁵	2.71 x 10 ⁵
Probable maximum moment at the center of RBS M_{pr} (kN.m)	590	590	207	104
Maximum moment at the face of the column M_{cf} (kN.m)	635	635	223	112
Maximum moment at the centerline of column M_c (kN.m)	701	701	251	130
V_h (kN)	157	157	66	41
$\frac{M_{cf}}{M_{pe}}$	0.75	0.75	0.70	0.72
Design moment at the center of RBS M_f (kN.m)	482	482	168	84

Table A.15: Design of column for moderately ductile MRF MD1

Limit state	Check for overall member strength	Check for lateral torsional buckling
F_y (kPa)	350	350
C_f (kN)	865	865
C_r (kN)	7017	4634
M_f (kN.m) (1.07)	628	628
M_x (kN.m)	2145	2145
U_1	1.0	1.0
$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x}$	0.37	0.44

Table A.16: Design of column for moderately ductile MRF MD2

Limit state	Check for overall member strength	Check for lateral torsional buckling
F_y (kPa)	350	350
C_f (kN)	830	830
C_r (kN)	4043	2169
M_f (kN.m) (1.19)	540	540
M_x (kN.m)	913	904
U_1	1.0	1.0
$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x}$	0.77	0.90

Table A.17: Design of column for limited ductility MRF LD1

Limit state	Check for overall member strength	Check for lateral torsional buckling
F_y (kPa)	350	350
C_f (kN)	903	903
C_r (kN)	7739	5212
M_f (kN.m) (1.03)	701	701
M_x (kN.m)	2400	2400
U_1	1.0	1.0
$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x}$	0.36	0.42

Table A.18: Design inter-storey drift

Type of MRF	First floor	Second floor	Third floor	Roof
Moderately ductile MD1 MRF	2.52	2.39	2.45	2.53
Moderately ductile MD2 MRF	3.01	2.98	2.78	2.80
Limited ductility LD1 MRF	2.25	2.01	2.23	2.26

Figure A.1: Plan of the building and locations of moment resisting frames

Figure A.2: Seismic hazard zone map (Geological Survey of Canada 2015)

Figure A.3: Gravity load distribution on the moment resisting frame

Figure A.4: Design spectrum for site class D at Powell River, British Columbia (NBCC 2015)

Figure A. 5: Lateral load distribution on single moderately ductile MD1 MRF

Figure A.6: Lateral load distribution on single moderately ductile MD2 MRF

Figure A.7: Lateral load distribution on single limited ductility LD1 MRF

Figure A.8: Details of reduced beam section

Figure A.9: Member sizes for MRFs MD1, MD2 and LD1

Appendix B: Design of moment resisting frame with RBS – preliminary model

Two single storey moderately ductile moment resisting frames (MRF) with reduced beam section (RBS) that have different flange width cut ratios have been considered in investigating the effect of cut ratio. The moment resisting frame from a single storey building has the same storey height, bay width, overall plan dimensions and placement of moment resisting frame similar to that of the four storey building shown in Fig. A.1. One of the single storey moment resisting frame is also being used to conduct the mesh sensitivity analysis to determine the optimum mesh configuration to carry out analyses of the four storey frame.

Here, the same method similar to Appendix A has been adopted to determine acting loads and member sizes. Similar to the four storey building, the single storey building is an office building located on site class D in Powell River, British Columbia with design spectrum represented in Fig. A.3. The single storey moment resisting frame with reduced beam section has been designed according to the loadings and procedures as outlined in Appendix A for the four storey moment resisting frame. Since it is a single storey building, only dead load and snow load are considered to act on the roof. Tables A.1 and A.2 show the breakdown of the total snow load and dead load on the roof. Only 25% of the 1.92 kPa snow load, that is 0.48 kPa, contributes to the seismic weight. Table B.1 shows the calculation of total seismic weight acting on the roof. The roof area is 36 m x 36 m (1296 m²) with the seismic load 2786 kN. The height of the building is 4 m, and thus the initial fundamental time period is 0.24s according to the formula $0.085h^{3/4}$ in NBCC 2015 Clause

4.1.8.

For the design of RBS, the maximum depth of the RBS cut should range between 20% to 50% of the total beam flange width according to the guidelines in Moment Connections for Seismic Applications (CISC 2014). Two single storey MRFs with different RBS have been considered to evaluate the effect of cut ratio. One frame having the cut ratio closer to the upper limit of 50% is designated as PM1, and the other closer to the lower limit of 20% as PM2.

The base shear for these models is calculated according to the procedure described in Section A.2 in Appendix A. For a moderately ductile moment resisting frame, the R_d and R_0 values are 3.5 and 1.5 respectively. The time period for PM1 is 1.24s and for PM2 is 1.0s for the final design with the seismic shear force shown in the seismic shear force calculation in Table B.2 as 65 kN and 71 kN respectively. Member sizes for the frame are shown in Tables B.3 and B.4, and Figs. B.4 to B.7, with a beam of W360x39 and column of W410x54 for PM1, and beam of W360x51 and column of W460x60 for PM2. According to the Eqs. A.7 to A.9, the final loadings including gravity load, seismic shear force and notional loads on the frames are represented in Figs. B.1 to B.3. The reduced beam section is designed according to the guidelines of Moment Connections for Seismic Applications (CISC 2014) as described in Section A.3 in Appendix A. Dimensions and design summary of RBS for PM1 are shown in Table B.5 and Fig. B.5, and for PM2 in Table B.6 and Fig. B.7. The RBS for PM1 has a maximum reduction of 46% in the flange width, and PM2 32% maximum reduction. Summaries of the column design for PM1 and PM2 are shown in Tables B.7 and B.8 with the frames having the design inter-storey drifts of 3.68% for PM1 and 3.45% for PM2.

Table B.1: Total seismic weight on the roof

	Roof
Area	36 m x 36 m
Snow load (kPa) (25% of total snow load)	0.48
Dead load (kPa)	1.67
Total dead + snow load (kPa)	2.15
Total seismic weight (kN)	2786

Table B.2: Seismic shear force on the roof of moment resisting frame

Level	Area (m ²)	$\frac{W_t}{Area}$	Weight W_i	Height H_i (m)	$W_i H_i$	$\frac{W_i H_i}{\sum W_i H_i}$	Shear force, V (kN)	
							PM1	PM2
Roof	1296	2.15	2786	4	11144	1.0	65	71

Table B.3: Sectional properties of structural members for the preliminary study model

PM1

	Column (W410x54)	Roof beam (W360x39)
Depth of cross-section d (mm)	403	353
Width of flanges b (mm)	177	128
Thickness of flanges t (mm)	10.9	10.7
Thickness of web w (mm)	7.5	6.5
Moment of inertia I_x (mm ⁴)	186x10 ⁶	102x10 ⁶
Section modulus Z_x (mm ³)	1050x10 ³	662 x 10 ³
Moment on the beam at face of the column M_f (kN.m)	-	206

Table B.4: Sectional properties of structural members for the preliminary study model

PM2

	Column (W460x60)	Roof beam (W360x51)
Depth of cross-section d (mm)	455	355
Width of flanges b (mm)	153	171
Thickness of flanges t (mm)	13.3	11.6
Thickness of web w (mm)	8.0	7.2
Moment of inertia I_x (mm ⁴)	255x10 ⁶	141x10 ⁶
Section modulus Z_x (mm ³)	1280x10 ⁶	894x10 ³
Moment on the beam at face of the column M_f (kN.m)	-	302

Table B.5: Design summary of RBS for PM1

Plastic section modulus at the center of RBS, Z_e , (mm^3)	0.43×10^6
Maximum probable moment at center of RBS, M_{pr} (kN.m)	188
Maximum probable moment at the face of column, M_{cf} (kN.m)	204
Maximum probable moment at the centerline of column, M_c (kN.m)	218
Shear force at the center of RBS, V_h (kN)	79
$\frac{M_{cf}}{M_{pe}}$	0.79
Design moment of RBS (kN.m)	138
$\sum M_{rc}$, (kN.m)	557
$\sum \left(M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right)$, (kN.m)	224

Table B.6: Design summary of RBS for PM2

Plastic section modulus at the center of RBS, Z_e , (mm ³)	0.68 x 10 ⁶
Maximum probable moment at center of RBS, M_{pr} (kN.m)	287
Maximum probable moment at the face of column, M_{cf} (kN.m)	305
Maximum probable moment at the centerline of column, M_c (kN.m)	323
Shear force at the center of RBS, V_h (kN)	84
$\frac{M_{cf}}{M_{pe}}$	0.88
Design moment of RBS (kN.m)	213
$\sum M_{rc}$, (kN.m)	806
$\sum \left(M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right)$, (kN.m)	365

Table B.7: Design of column for the preliminary study model PM1

Limit state	Check for overall member strength	Check for lateral torsional buckling
F_y (kPa)	350	350
C_f (kN)	156	156
C_r (kN)	1789	495
M_f (kN.m)	218	218
M_x (kN.m)	331	302
U_1	1.0	1.0
$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x}$	0.64	0.80
Design inter-storey drift (%)	3.68	

Table B.8: Design of column for the preliminary study model PM2

Limit state	Check for overall member strength	Check for lateral torsional buckling
F_y (kPa)	350	350
C_f (kN)	191	191
C_r (kN)	2327	743
M_f (kN.m)	300	300
M_x (kN.m)	403	350
U_1	1.0	1.0
$\frac{C_f}{C_r} + \frac{0.85 U_1 M_f}{M_x}$	0.71	0.98
Design inter-storey drift (%)	3.45	

Figure B.1: Gravity load distribution on the moment resisting frame

Figure B.2: Lateral load distribution on preliminary model PM1

Figure B.3: Lateral load distribution on preliminary model PM2

Figure B.4: Dimensions and cross-section of final design of preliminary study model PM1

Figure B.5: Details of reduced beam section for the preliminary study model PM1

Figure B.6: Dimensions and cross-section of final design of preliminary study model PM2

Figure B.7: Details of reduced beam section for the preliminary study model PM2

Appendix C: Predicted bending moment at the center of RBS for MRFs

Table C.1: Predicted moment at the center of RBS for moderately ductile MRF (MD1) at first floor

ER	Material H					Material G				
	Stati, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Stati, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	529	561	6.0	1.02	1.08	522	555	6.3	1.00	1.07
2	534	568	6.4	1.03	1.09	525	562	7.0	1.01	1.08
3	543	597	9.9	1.04	1.15	528	591	11.9	1.02	1.14
4	571	618	8.2	1.10	1.19	541	600	10.9	1.04	1.15
5	584	623	6.7	1.12	1.20	549	613	11.7	1.06	1.18
6	529	558	5.5	1.02	1.07	520	549	5.6	1.00	1.06
7	535	575	7.5	1.03	1.11	527	569	8.0	1.01	1.09
8	567	615	8.5	1.09	1.18	530	588	10.9	1.02	1.13
9	536	567	5.8	1.03	1.09	520	557	7.1	1.00	1.07
10	529	560	5.9	1.02	1.08	519	553	6.6	1.00	1.06
11	528	558	5.7	1.02	1.07	519	550	6.0	1.00	1.06
Avg	544	582	6.9	1.05	1.12	527	572	8.4	1.01	1.10

Table C.2: Predicted moment at the center of RBS for moderately ductile MRF (MD1) second floor

ER	Material H					Material G				
	Static, M_s	Rate, M_r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M_s	Rate, M_r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	532	564	6.0	1.02	1.08	525	558	6.3	1.01	1.07
2	538	573	6.5	1.03	1.10	528	566	7.2	1.02	1.09
3	547	600	9.7	1.05	1.15	532	599	12.6	1.02	1.15
4	580	621	7.1	1.12	1.19	544	607	11.6	1.05	1.17
5	590	625	5.9	1.13	1.20	553	618	11.8	1.06	1.19
6	534	563	5.4	1.03	1.08	526	559	6.3	1.01	1.08
7	541	583	7.8	1.04	1.12	529	571	7.9	1.02	1.10
8	574	619	7.8	1.10	1.19	534	593	11.0	1.03	1.14
9	541	572	5.7	1.04	1.10	523	559	6.9	1.01	1.08
10	535	566	5.8	1.03	1.09	521	555	6.5	1.00	1.07
11	532	562	5.6	1.02	1.08	520	551	6.0	1.00	1.06
Avg	549	586	6.7	1.06	1.13	530	576	8.6	1.02	1.11

Table C.3: Predicted moment at the center of RBS for moderately ductile MRF (MD1) at third floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	189	199	5.3	1.03	1.08	184	196	6.5	1.00	1.07
2	191	202	5.8	1.04	1.10	185	199	7.6	1.01	1.08
3	196	213	8.7	1.07	1.16	188	212	12.8	1.02	1.15
4	206	219	6.3	1.12	1.19	195	218	11.8	1.06	1.18
5	209	221	5.7	1.14	1.20	197	220	11.7	1.07	1.20
6	189	199	5.3	1.03	1.08	185	197	6.5	1.01	1.07
7	195	207	6.2	1.06	1.13	187	203	8.6	1.02	1.10
8	203	217	6.9	1.10	1.18	190	211	11.1	1.03	1.15
9	192	202	5.2	1.04	1.10	184	197	7.1	1.00	1.07
10	189	199	5.3	1.03	1.08	183	195	6.6	0.99	1.06
11	187	197	5.3	1.02	1.07	183	194	6.0	0.99	1.05
Avg.	195	207	6.0	1.06	1.12	187	204	8.8	1.02	1.11

Table C.4: Predicted moment at the center of RBS for moderately ductile MRF (MD1) at roof

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	98	103	5.1	1.02	1.07	96	102	6.8	0.99	1.06
2	101	107	5.4	1.05	1.11	96	104	7.8	1.00	1.08
3	103	111	7.6	1.07	1.16	97	109	12.4	1.01	1.14
4	106	112	5.7	1.10	1.17	100	112	11.8	1.04	1.16
5	109	115	5.5	1.14	1.20	101	114	12.9	1.05	1.19
6	100	105	5.0	1.04	1.09	97	104	6.7	1.01	1.08
7	102	108	5.9	1.06	1.13	98	106	8.2	1.02	1.10
8	105	111	5.7	1.09	1.16	98	109	11.2	1.02	1.14
9	99	104	5.1	1.03	1.08	96	102	6.8	0.99	1.06
10	98	103	5.2	1.02	1.07	95	101	6.3	0.99	1.05
11	96	101	5.3	1.00	1.06	94	100	6.0	0.98	1.04
Avg.	102	107	5.6	1.06	1.12	97	106	8.8	1.01	1.10

Table C.5: Predicted moment at the center of RBS for moderately ductile MRF (MD2) at first floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	465	498	7.1	1.01	1.08	461	494	7.2	1.00	1.07
2	469	505	7.7	1.02	1.10	463	500	8.0	1.01	1.09
3	472	523	10.8	1.03	1.14	464	520	12.1	1.01	1.13
4	476	526	10.5	1.03	1.14	470	524	11.5	1.02	1.14
5	485	538	10.9	1.05	1.17	475	531	11.8	1.03	1.15
6	465	496	6.7	1.01	1.08	462	494	6.9	1.00	1.07
7	471	510	8.3	1.02	1.11	466	505	8.4	1.01	1.10
8	475	525	10.5	1.03	1.14	469	521	11.1	1.02	1.13
9	466	499	7.1	1.01	1.08	462	496	7.4	1.00	1.08
10	462	496	7.4	1.00	1.08	460	495	7.6	1.00	1.08
11	460	492	7.0	1.00	1.07	458	491	7.2	1.00	1.07
Avg.	470	510	8.5	1.02	1.11	465	506	9.0	1.01	1.10

Table C.6: Predicted moment at the center of RBS for moderately ductile MRF (MD2) at second floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	451	482.5	7.0	1.00	1.07	446	480	7.6	0.99	1.07
2	452	485	7.3	1.01	1.08	448	483	7.8	1.00	1.08
3	460	510	10.9	1.02	1.14	450	508	12.9	1.00	1.13
4	468	511	9.2	1.04	1.14	456	510	11.8	1.02	1.14
5	473	519	9.7	1.05	1.16	461	516	11.9	1.03	1.15
6	451	480	6.4	1.00	1.07	448	479	6.9	1.00	1.07
7	457	493	7.9	1.02	1.10	453	491	8.4	1.01	1.09
8	459	503	9.6	1.02	1.12	453	502	10.8	1.01	1.12
9	452	483	6.9	1.01	1.08	447	480	7.4	1.00	1.07
10	450	482	7.1	1.00	1.07	445	479	7.6	0.99	1.07
11	449	480	6.9	1.00	1.07	444	477	7.4	0.99	1.06
Avg.	457	494	8.1	1.02	1.10	450	491	9.1	1.00	1.09

Table C.7: Predicted moment at the center of RBS for moderately ductile MRF (MD2) at third floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	186	198	6.5	1.03	1.09	183	197	7.7	1.01	1.09
2	188	201	6.9	1.04	1.11	183	198	8.2	1.01	1.09
3	192	210	9.4	1.06	1.16	184	208	13.0	1.02	1.15
4	193	211	9.3	1.07	1.17	186	210	12.9	1.03	1.16
5	198	216	9.1	1.09	1.19	188	213	13.3	1.04	1.18
6	188	198	5.3	1.04	1.09	184	197	7.1	1.02	1.09
7	190	202	6.3	1.05	1.12	185	201	8.6	1.02	1.11
8	194	209	7.7	1.07	1.15	186	207	11.3	1.03	1.14
9	187	200	7.0	1.03	1.10	183	197	7.7	1.01	1.09
10	186	199	7.0	1.03	1.10	182	196	7.7	1.01	1.08
11	185	198	7.0	1.02	1.09	181	195	7.7	1.00	1.08
Avg.	190	204	7.4	1.05	1.13	184	202	9.8	1.02	1.11

Table C.8: Predicted moment at the center of RBS for moderately ductile MRF (MD2) at roof

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	86	92	6.4	1.01	1.08	84	91	8.0	0.99	1.07
2	86	92	7.0	1.01	1.08	84	92	8.5	0.99	1.08
3	87	97	11.5	1.02	1.14	85	95	12.2	1.00	1.12
4	89	98	10.1	1.05	1.15	86	98	14.0	1.01	1.15
5	92	100	9.0	1.08	1.18	87	100	14.9	1.02	1.18
6	87	92	5.3	1.02	1.08	85	91	7.1	1.00	1.07
7	88	93	6.1	1.04	1.10	86	93	8.8	1.01	1.09
8	90	96	6.7	1.06	1.13	86	95	10.5	1.01	1.12
9	86	92	6.5	1.01	1.08	85	90	6.9	0.99	1.06
10	86	91	6.5	1.01	1.07	84	90	7.1	0.99	1.06
11	84	89	6.0	0.99	1.05	83	89	6.7	0.98	1.04
Avg.	87	94	8.0	1.02	1.11	85	93	9.4	1.00	1.09

Table C.9: Predicted moment at the center of RBS for limited ductility MRF (LD1) at first floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	610	645	5.7	1.03	1.09	601	636	5.8	1.02	1.08
2	618	656	6.1	1.05	1.11	611	652	6.7	1.04	1.11
3	630	677	7.5	1.07	1.15	617	668	8.3	1.05	1.13
4	636	679	6.8	1.08	1.15	625	675	8.0	1.06	1.14
5	642	687	7.0	1.09	1.16	636	683	7.4	1.08	1.16
6	613	644	5.1	1.04	1.09	603	636	5.5	1.02	1.08
7	628	669	6.5	1.06	1.13	617	661	7.1	1.05	1.12
8	632	675	6.8	1.07	1.14	621	670	7.9	1.05	1.14
9	615	649	5.5	1.04	1.10	606	644	6.3	1.03	1.09
10	611	645	5.6	1.04	1.09	600	638	6.3	1.02	1.08
11	607	641	5.6	1.03	1.09	599	635	6.0	1.02	1.08
Avg.	622	661	6.3	1.05	1.12	612	654	6.9	1.04	1.11

Table C.10: Predicted moment at the center of RBS for limited ductility MRF (LD1) at second floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	618	649	5.0	1.05	1.10	603	639	6.0	1.02	1.08
2	622	658	5.8	1.05	1.12	612	656	7.2	1.04	1.11
3	635	679	6.9	1.08	1.15	618	672	8.7	1.05	1.14
4	642	684	6.5	1.09	1.16	628	680	8.3	1.06	1.15
5	648	692	6.8	1.10	1.17	639	690	8.0	1.08	1.17
6	620	646	4.2	1.05	1.09	606	641	5.8	1.03	1.09
7	634	670	5.7	1.07	1.14	620	666	7.4	1.05	1.13
8	636	678	6.6	1.08	1.15	624	674	8.0	1.06	1.14
9	619	652	5.3	1.05	1.11	610	648	6.2	1.03	1.10
10	615	648	5.4	1.04	1.10	603	642	6.5	1.02	1.09
11	611	643	5.2	1.04	1.09	600	637	6.2	1.02	1.08
Avg.	627	664	5.9	1.06	1.13	615	659	7.2	1.04	1.12

Table C.11: Predicted moment at the center of RBS for limited ductility MRF (LD1) at third floor

ER	Material H					Material G				
	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M _s	Rate, M _r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	216	226	4.6	1.04	1.09	211	224	6.2	1.02	1.08
2	219	230	5.0	1.06	1.11	213	228	7.0	1.03	1.10
3	223	237	6.3	1.08	1.14	216	235	8.8	1.04	1.14
4	231	245	6.1	1.12	1.18	223	242	8.5	1.08	1.17
5	240	255	6.3	1.16	1.23	231	250	8.2	1.12	1.21
6	217	226	4.1	1.05	1.09	211.5	224	5.9	1.02	1.08
7	222	234	5.4	1.07	1.13	215.5	232	7.7	1.04	1.12
8	225	239	6.2	1.09	1.15	218	237	8.7	1.05	1.14
9	217	228	5.1	1.05	1.10	213	226	6.1	1.03	1.09
10	214	225	5.1	1.03	1.09	208	222	6.7	1.00	1.07
11	210	220	4.8	1.01	1.06	205	218	6.3	0.99	1.05
Avg.	221	233	5.4	1.07	1.13	215	231	7.4	1.04	1.12

Table C.12: Predicted moment at the center of RBS for limited ductility MRF (LD1) at roof

ER	Material H					Material G				
	Static, M_s	Rate, M_r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$	Static, M_s	Rate, M_r	% increase over static	$\frac{M_s}{M_{pr}}$	$\frac{M_r}{M_{pr}}$
1	104	108	3.8	1.00	1.04	101	107.5	6.4	0.97	1.03
2	105	110	4.8	1.01	1.06	102	109	6.9	0.98	1.05
3	108	114.5	6.0	1.04	1.10	103	112.3	9.0	0.99	1.08
4	111	117.5	5.9	1.07	1.13	106.5	116	8.9	1.02	1.12
5	114	121	6.1	1.10	1.16	109	118	8.3	1.05	1.13
6	104.2	109	4.6	1.00	1.05	102	108	5.9	0.98	1.04
7	106	111.5	5.2	1.02	1.07	103	111	7.8	0.99	1.07
8	109	116	6.4	1.05	1.12	105	114.3	8.9	1.01	1.10
9	105	110	4.8	1.01	1.06	102	108.5	6.4	0.98	1.04
10	104	109	4.8	1.00	1.05	101	108	6.9	0.97	1.04
11	103	107.5	4.4	0.99	1.03	100	106.4	6.4	0.96	1.02
Avg.	107	112	4.7	1.03	1.08	103	111	7.8	0.99	1.07

Appendix D: Effects of higher modes on moment resisting frame

A structure can have more than one dominant mode. Higher modes can have significant impact on the behavior of a structure under seismic excitation. Therefore, it is necessary to compare the results of the first mode (dominant) with the higher modes to understand its significance on the behavior of the structure. In this case, results obtained from the second mode are compared to that from the first mode to investigate effects of higher modes. For the moderately ductile MRF (MD1) the time period for first and second modes are 1.45s and 0.12s respectively. Similarly, for the limited ductility MRF (LD1) the time period for the first mode is 1.24s and for the second mode is 0.09s.

The first step in conducting a non-linear dynamic analysis subjected to earthquake excitation is to scale the earthquake records with respect to the second mode. Here, the spectral acceleration of the entire suite of earthquake records is scaled to match the design spectrum over a time period range of 0.02s to 1.5s for the moderately ductile MRF (MD1) and 0.01s to 1.5s for limited ductility MRF (LD1). These time period ranges are calculated from the Eqs. (5.2) and (5.3). As shown in Figs. D.1 and D.2, the mean spectrum obtained for scaling of moderately ductile MRF (MD1) and limited ductility MRF (LD1) are plotted to compare their correlation with the design spectrum. Both moderately ductile MRF (MD1) and limited ductility MRF (LD1) are subjected to these scaled earthquake records to conduct the seismic analyses. Results of the analyses are compared to that for the analyses with the earthquake records scaled for the first mode. Comparisons of the moment at the center of RBS and inter-storey drift are shown in Tables D.1 and D.2, and Figs. D.3 and D.4 for moderately ductile MRF (MD1), and Tables D.3 and D.4, and Figs. D.5 and

D.6 for limited ductility MRF (LD1).

Results in Table D.1 for the moment at the center of RBS of the moderately ductile MRF show that the moment decreases by approximately 18% and 22% for materials H and G using the static material properties with scaling for mode 2 instead of mode 1. The decrease in the bending moment at the center of RBS increases to an average of 18.5% and 23% with strain rate dependent material properties for materials H and G. As shown in Fig. D.3, there is little change in the percentage decrease in bending moment for all floors with the second floor level having the smallest decrease. Similarly with strain rate dependent material properties as shown in Fig. D.4, the percentage decrease at the second floor level is the smallest, but the difference with other floors is small. Table D.3 shows that there is a corresponding decrease in the inter-storey drift using earthquake records scaled for mode 2 instead of mode 1. The average percentage drop in the inter-storey drift from mode 1 to mode 2 are 9.3% and 9.4% for materials H and G with static material properties, and 9.4% and 9.5% with strain rate dependent material properties.

Tables D.4 shows the percentage decrease in the bending moment at the center of RBS for limited ductility MRF (LD1) with earthquake records scaled for mode 2 instead of mode 1 decreases at an average decrease of 14% and 17% using the static material properties for materials H and G. The corresponding average decrease with rate dependent material properties are 15% and 18%. As shown in Fig. D.5 with static material properties and Fig. D.6 with strain rate dependent material properties for MRF LD1, there is little change in the percentage decrease in the bending moment for all floors except a smaller decrease at the second floor level. The percentage decrease in bending moment for MRF LD1 shows similar trends to that of MRF MD1. Table D.4 shows the comparisons of the

inter-storey drift for limited ductility MRF (LD1) with earthquake records scaled for mode 2 instead of mode 1. It can be seen that there is an average decrease of 11.5% and 11.3% from mode 1 to mode 2 for materials H and G with static material properties, and corresponding decrease of 11.7% and 11.5% with strain rate dependent material properties.

Tables D.1 to D.4 clearly show that the bending moment at the center of RBS and inter-storey drift are much smaller on an average of close to at least 10% with earthquake records scaled for mode 2 time period instead of mode 1. Therefore, the rest of the study has focused on effects due to earthquake records scaled only for mode 1.

Table D.1: Mean predicted maximum moment at the center of RBS with static and strain rate dependent material properties for moderately ductile MRF (MD1)

Level	Material	Mean predicted maximum moment at the center of RBS (kN.m)					
		Static, M_s			Rate, M_r		
		Mode 1	Mode 2	% decrease mode 2 below mode 1	Mode 1	Mode 2	% decrease mode 2 below mode 1
First floor	Material H	544	461	18.0	582	491	18.5
	Material G	527	433	21.7	572	465	23.0
Second floor	Material H	549	467	17.6	586	496	18.1
	Material G	530	437	21.3	576	471	22.3
Third floor	Material H	195	165	18.2	207	175	18.3
	Material G	187	153	22.2	204	166	22.9
Roof	Material H	102	86	18.6	107	90	18.9
	Material G	97	79	22.8	106	86	23.3
Average	Material H	348	295	18.1	371	313	18.5
	Material G	335	276	22.0	365	297	22.9

Table D.2: Mean predicted inter-storey drift with static and strain rate dependent material properties for moderately ductile MRF (MD1)

Level	Material	Mean predicted inter-storey drift, %					
		Static, M_s			Rate, M_r		
		Mode 1	Mode 2	% decrease mode 2 below mode 1	Mode 1	Mode 2	% decrease mode 2 below mode 1
First floor	Material H	2.29	2.09	9.6	2.26	2.05	10.2
	Material G	2.27	2.07	9.7	2.27	2.07	9.7
Second floor	Material H	1.91	1.77	7.9	1.89	1.76	7.4
	Material G	1.90	1.76	7.9	1.89	1.75	8.0
Third floor	Material H	2.24	2.05	9.3	2.23	2.05	8.8
	Material G	2.24	2.06	8.7	2.23	2.06	8.3
Roof	Material H	2.36	2.14	10.3	2.36	2.13	10.8
	Material G	2.36	2.12	11.3	2.36	2.11	11.8
Average	Material H	2.20	2.01	9.3	2.19	2.00	9.3
	Material G	2.19	2.00	9.4	2.19	2.00	9.5

Table D.3: Mean predicted maximum moment at the center of RBS with static and strain rate dependent material properties for limited ductility MRF (LD1)

Level	Material	Mean predicted maximum moment at the center of RBS (kN.m)					
		Static, M_s			Rate, M_r		
		Mode 1	Mode 2	% decrease mode 2 below mode 1	Mode 1	Mode 2	% decrease mode 2 below mode 1
First floor	Material H	622	547	13.7	661	577	14.6
	Material G	612	524	16.8	654	558	17.2
Second floor	Material H	627	555	13.0	664	585	13.5
	Material G	615	531	15.8	659	566	16.4
Third floor	Material H	221	194	13.9	233	202	15.3
	Material G	215	184	17.2	231	196	17.9
Roof	Material H	107	94	14.3	112	97	15.5
	Material G	103	87	18.4	111	94	18.7
Average	Material H	394	348	13.7	418	365	14.7
	Material G	386	332	17.1	414	354	17.6

Table D.4: Mean predicted inter-storey drift with static and strain rate dependent material properties for limited ductility MRF (LD1)

Level	Material	Mean predicted inter-storey drift, %					
		Static, M_s			Rate, M_r		
		Mode 1	Mode 2	% decrease mode 2 below mode 1	Mode 1	Mode 2	% decrease mode 2 below mode 1
First floor	Material H	1.87	1.63	14.7	1.87	1.63	14.7
	Material G	1.87	1.62	15.4	1.86	1.62	14.8
Second floor	Material H	1.48	1.35	9.6	1.48	1.35	9.6
	Material G	1.46	1.35	8.2	1.46	1.35	8.2
Third floor	Material H	2.07	1.88	10.1	2.07	1.87	10.7
	Material G	2.07	1.88	10.1	2.07	1.88	10.1
Roof	Material H	2.11	1.89	11.6	2.12	1.90	11.6
	Material G	2.11	1.89	11.6	2.12	1.88	12.8
Average	Material H	1.88	1.69	11.5	1.89	1.69	11.7
	Material G	1.88	1.69	11.3	1.88	1.68	11.5

Figure D.1: Mean spectral acceleration of the suite of scaled earthquake records for mode 2 of moderately ductile MRF (MD1)

Figure D.2: Mean spectral acceleration of the suite of scaled earthquake records for mode 2 of limited ductility MRF (LD1)

Figure D.3: Percentage (%) decrease in the bending moment at the center of RBS for mode 2 with respect to mode 1 with static material properties for moderately ductile MRF (MD1)

Figure D.4: Percentage (%) decrease in the bending moment at the center of RBS for mode 2 with respect to mode 1 with strain rate dependent material properties for moderately ductile MRF (MD1)

Figure D.5: Percentage (%) decrease in the bending moment at the center of RBS for mode 2 with respect to mode 1 with static material properties for limited ductility MRF (LD1)

Figure D.6: Percentage (%) decrease in the bending moment at the center of RBS for mode 2 with respect to mode 1 with strain rate dependent material properties for limited ductility MRF (LD1)

Appendix E: Comparisons of moderately ductile moment resisting frame with fixed and pinned supports

In the study, all the MRFs considered are pinned supported. Thus, a brief study on the MRF with fixed supports has been carried out. A four storey moderately ductile MRF with fixed supports, designated as MD3, has been designed. The frame has been designed for strength and to meet the inter-storey drift limit of less than 2.5%.

The frame is assumed to be located on the same site location as MD1 and MD2. Thus, snow load, dead load, live load, seismic weight, and design spectrum for MD3 is taken from Tables A.1 to A.4 of Appendix A. Final member sizes for MD3 are shown in Table E.1 and Fig. E.1. Design summary and dimensions of RBS on each storey are listed in Tables E.2 and E.3. The final fundamental time period of this frame is 1.26s. Figure E.2 shows the seismic load and other lateral loads acting on MD3. The design inter-storey drifts are shown in Table E.4.

The same suite of earthquake records chosen for MD1 are used for MD3 in the analyses. Details of these earthquake records are listed in Table 5.1. This suite of earthquake records are scaled according to the time period of MD3 at 1.26s. The procedure in Section 5.2 for scaling of seismic records has been used. From the suite of 11 earthquake records, only the first five are selected for performing seismic analyses of MD3. Material properties for material H in Section 3.1 have been used in the analyses. Comparisons are made between results of MD1 and MD3 to investigate effects of support type of pinned and fixed.

Table E.5 shows the design and average predicted inter-storey drifts for MD1 and MD3. It can be seen that there is very little difference between results with static and strain

rate dependent material properties. In both MD1 and MD3, the predicted inter-storey drift is lower than that for the design and with the second floor having the largest difference. Overall, the predicted inter-storey drift for MD3 is lower than the design value by an additional 0.1% compared to MD1.

Comparisons of the mean predicted maximum moment at the center of RBS for MD1 and MD3 are shown in Tables E.6 and E.7. Overall, there is no significant difference between MD1 and MD3 in the rate of increase of the predicted maximum moment with strain rate dependent over that with static material properties. Both MD1 and MD3 also have comparable maximum ratio of the predicted maximum moment over the probable moment at the center of RBS. Thus it can be expected that findings on the bending moment at the center of RBS and inter-storey drift for pinned supports MRFs can be extended to fixed supported MRFs.

A suite of 11 earthquake records has been scaled to match the fundamental time period of MD3 on the design spectrum. However, analyses have been conducted using only the first five earthquake records in order to reduce the total computing and data extraction time. The difference between the spectral acceleration of the design spectrum and the mean spectrum at the time period 1.26s from the scaled first five earthquake records is 1.5%. For the suite of 11 scaled earthquake records, this difference in spectral acceleration is 1.0%. As both the differences are comparable, it can be expected the mean predicted results using the scaled first five earthquake records to be comparable to using the entire suite of scaled earthquake records.

Table E.1: Sectional properties of structural members for moderately ductile MRF MD3

	Column W610x92	First floor beam W360x39	Second floor beam W360x33	Third floor beam W310x28	Roof beam W250x25
Depth of cross-section d (mm)	603	353	349	309	257
Width of flanges b (mm)	179	128	127	102	102
Thickness of flanges t (mm)	15.0	10.7	8.5	8.9	8.4
Thickness of web w (mm)	10.9	6.5	5.8	6.0	6.1
Moment of inertia I_x (mm ⁴)	651×10^6	102×10^6	82.7×10^6	54.3×10^6	34.2×10^6
Section modulus Z_x (mm ³)	2530×10^3	662×10^3	542×10^3	407×10^3	307×10^3
Design moment on the beam at face of the column M_f (kN.m)	-	199	165	124	90

Table E.2: Design summary of RBS for limited ductility MRF MD3

	First floor beam	Second floor beam	Third floor beam	Roof beam
Plastic section modulus at the center of RBS Z_e (mm ³)	0.45x10 ⁶	0.38x10 ⁶	0.27x10 ⁶	0.20x10 ⁶
Probable maximum moment at the center of RBS M_{pr} (kN.m)	190	160	115	85
Maximum moment at the face of the column M_{cf} (kN.m)	201	170	124	92
Maximum moment at the centerline of column M_c (kN.m)	227	193	142	107
V_h (kN)	61	54	44	37
$\frac{M_{cf}}{M_{pe}}$	0.79	0.82	0.79	0.77
Design moment at the center of RBS M_f (kN.m)	140	119	85	62

Table E.3: Dimensions of RBS for moderately ductile MRF MD3

Floor level	Transition zone a (mm)	Length of RBS s (mm)	Depth of RBS cut c (mm)
First floor beam	65	230	29.0
Second floor beam	65	230	28.0
Third floor beam	75	260	25.0
Roof floor beam	75	210	25.5

Table E.4: Design inter-storey drift for MD3

Floor Level	Inter-storey drift (%)
First	2.35
Second	2.19
Third	2.28
Roof	2.36

Table E.5: Comparisons of design and average predicted inter-storey drifts with static and strain rate dependent material properties at each floor level for MD1 and MD3

MRF	MD1			MD3		
Floor level	%, Design	%, Static	%, Rate	%, Design	%, Static	%, Rate
First	2.52	2.42	2.45	2.35	2.10	2.11
Second	2.39	2.07	2.07	2.19	1.78	1.78
Third	2.45	2.33	2.34	2.28	2.01	2.00
Roof	2.53	2.45	2.47	2.36	2.09	2.09

Table E.6: Mean predicted maximum moment at center of RBS with static and strain rate dependent material properties for MD1 and MD3

Bending moment	MD1			MD3		
Floor level	Static (M_s)	Rate (M_r)	$\frac{M_r}{M_s}$	Static (M_s)	Rate (M_r)	$\frac{M_r}{M_s}$
First	544	582	1.07	199	215	1.08
Second	549	586	1.07	170	183	1.08
Third	195	207	1.06	118	127	1.08
Roof	102	107	1.05	89	94	1.06

Table E.7: Percentage change in the mean predicted maximum moment at the center of RBS compared to probable moment with static and strain rate dependent material properties for MD1 and MD3

Floor Level	Static $\left(\frac{M_s}{M_{pr}}\right)$			Rate-dependent $\left(\frac{M_r}{M_{pr}}\right)$		
	MD1	MD3	% Change of MD1 over MD3	MD1	MD3	% Change of MD1 over MD3
First	1.06	1.05	-0.95	1.14	1.13	-0.88
Second	1.07	1.06	-0.94	1.15	1.14	-0.88
Third	1.08	1.03	-4.85	1.15	1.10	-4.55
Roof	1.08	1.05	-2.86	1.14	1.11	-2.70

Figure E.1: Member sizes of moderately ductile MRF (MD3)

Figure E.2: Lateral load distribution on a single moderately ductile MRF (MD3)