SEISMIC PERFORMANCE
OF STEEL MOMENT-RESISTING FRAMES WITH
NONLINEAR REPLACEABLE LINKS

By:

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A thesis submitted in conformity with the requirements
for the degree of Master of Applied Science
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This thesis presents the development and the seismic performance evaluation of steel MRFs with nonlinear replaceable links. Although existing MRFs can provide life safety during a design level earthquake, they are expected to sustain significant damage at the locations of flexural yielding fuses in the beams. The design of the fuse is also interlinked with the design of the beam, often resulting in over-design. These drawbacks can be mitigated by introducing replaceable links at the locations of expected inelastic action.

Four full-scale beam-to-column subassemblages with two link types were tested under cyclic loading: i) double channels with bolted web connections, ii) W-sections with bolted end plate connections. The experiments demonstrated that MRFs with replaceable links can provide strength and ductility equivalent to existing MRFs. Finite element models were then developed to capture the observed experimental responses, including local buckling, bolt slipping, and bolt bearing. Finally, preliminary design guidelines were proposed.
ACKNOWLEDGEMENT

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Lastly but most importantly, I would like to thank my parents for providing excellent opportunities for me at the cost of their own comfort, and for always being there for me.
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LIST OF SYMBOLS

\( a \) distance from the column face to the RBS or to the link connection; height of column panel zone

\( A_b \) cross-sectional area of a bolt based on its nominal diameter

\( b \) width of flange; width of column panel zone

\( b_{beam} \) beam flange width

\( B_r \) factored bearing resistance

\( b_p \) end plate width

\( c \) depth of cut at the center of the reduced beam section

\( C \) nondimensional coefficient relating individual bolt capacity to the capacity of an eccentric bolted connection

\( C_f \) compressive force in a member under factored load

\( C_y \) axial compressive load at yield stress

\( C_{pr} \) factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions

\( d \) overall depth of a section; distance between displacement transducers measuring connection rotation in double channel link subassemblage

\( D \) dead load

\( d_b \) depth of beam; bolt diameter

\( d_c \) depth of column

\( d_{link} \) depth of replaceable link

\( E_{EPP} \) energy dissipated by an elastic-perfectly-plastic specimen during one cycle

\( E_{norm} \) energy dissipated by the test specimen, normalized by equivalent \( E_{EPP} \) for one cycle

\( E_{specimen} \) energy dissipated by the test specimen during one cycle

\( E \) elastic modulus of steel (200000 MPa assumed); earthquake loads and effects

\( F_b \) point load applied on the beam during experiment

\( F_t \) concentrated lateral load at the top floor
List of Symbols

$F_u$ specified minimum tensile strength

$F_x'$ first order distributed lateral forces at each storey

$F_{x,final}$ distributed lateral forces at each storey including notional load, torsional effects, and second order effects

$F_y$ specified minimum yield strength

$F_{yc}$ specified minimum column yield strength

$F_{xp}$ specified minimum end plate yield strength, taken as 250 MPa

$g$ horizontal gauge between bolts

$h$ clear depth of web between flanges

$h_i$ distance from centerline of a compression flange to the centerline of the inner tension bolt row; height at the $i^{th}$ storey measured from the ground

$h_o$ distance from centerline of a compression flange to the centerline of the outer tension bolt row

$h_s$ storey height

$h_x$ height at the $x^{th}$ storey from the ground

$I_g$ importance factor of structure

$k_{elastic}$ linear elastic stiffness of the test subassemblage

$L$ distance from the point of loading to the column centerline; live load

$L'$ distance from the plastic hinge center to the point of loading during experiment

$m$ number of shear planes in a bolted connection

$M_f$ factored moment demand

$M_c$ probable moment demand at column center due to plastic hinge formation

$M_{c,f}$ probable moment demand at column face due to plastic hinge formation; moment at column face calculated from measured $F_b$

$M_r$ factored moment resistance of a member of component

$M_{rc}'$ cross-sectional bending strength of column

$M_p$ plastic moment resistance

$M_{pb}$ plastic moment of a beam
List of Symbols

\( M_{pc} \)  
plastic moment of a column

\( M_{pr} \)  
probable maximum moment at plastic hinge

\( M_{pb, beam} \)  
plastic moment of a beam

\( M_{pl,link} \)  
plastic moment of a link

\( M_i \)  
factor accounting for higher mode effect

\( R \)  
bolt load at any given deformation

\( R_f \)  
the maximum bolt demand in an eccentric bolted connection, calculated from the factored demand on the connection

\( R_u \)  
ultimate bolt load

\( p_f \)  
distance from the outside of a beam tension flange to the nearest outside bolt row

\( P_f \)  
factored demand on an eccentric bolted connection

\( p_i \)  
distance from the outside of a beam tension flange to the nearest inside bolt row

\( R_d \)  
ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour, 5.0 for special moment frames

\( R_o \)  
overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure, 1.5 for special moment frames

\( R_y \)  
factor applied to \( F_y \) to estimate the probable yield stress, 1.1

\( s \)  
length of an RBS cut or length of a replaceable link; distance from the centerline of the inside tension bolt row to the edge of a yield line pattern

\( S \)  
snow load

\( S_a \)  
spectral acceleration

\( t \)  
plate thickness

\( T_a \)  
foundamental lateral period of a building calculated according to NBCC 4.1.8.11.3 a)

\( t_{\text{link}} \)  
thickness of link flange

\( t_p \)  
thickness of end plate

\( T \)  
lateral period of a structure

\( T_f \)  
tensile force in a member or component under factored load

\( T_r \)  
factored tensile resistance
**List of Symbols**

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<tr>
<td>$U_2$</td>
<td>factor accounting for P-Delta effects</td>
</tr>
<tr>
<td>$V$</td>
<td>total base shear calculated according to equivalent static force procedure</td>
</tr>
<tr>
<td>$V_d$</td>
<td>design base shear, larger of $\frac{V_e I_E}{K_d R_o}$ or $0.8 V$</td>
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<tr>
<td>$V_e$</td>
<td>elastic base shear of a structure obtained from a linear dynamic analysis</td>
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<tr>
<td>$V_h$</td>
<td>shear acting at plastic hinge location when plastic hinging occurs</td>
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<tr>
<td>$V_r$</td>
<td>factored shear resistance of column panel zone; factored shear resistance of a bolt</td>
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<tr>
<td>$V_{pz}$</td>
<td>probable shear demand on column panel zone due to plastic hinge formation</td>
</tr>
<tr>
<td>$w$</td>
<td>web thickness</td>
</tr>
<tr>
<td>$\bar{w}$</td>
<td>deadload of building plus 25% of snow load</td>
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<tr>
<td>$w'$</td>
<td>sum of thickness of column web plus doubler plates</td>
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<tr>
<td>$Z_b$</td>
<td>plastic modulus of beam</td>
</tr>
<tr>
<td>$Z_c$</td>
<td>plastic modulus of column</td>
</tr>
<tr>
<td>$Z_{link}$</td>
<td>plastic modulus of replaceable link</td>
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<tr>
<td>$\Delta$</td>
<td>bolt deformation and local deformation of connecting material</td>
</tr>
<tr>
<td>$\Delta_f$</td>
<td>relative first order lateral displacement of the storey due to factored loads</td>
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<td>$\delta_b$</td>
<td>imposed displacement on the beam during experiment</td>
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<tr>
<td>$\varepsilon$</td>
<td>true strain</td>
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<td>$\varepsilon_{nom}$</td>
<td>measured engineering strain</td>
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<td>$\varepsilon^{pl}$</td>
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<td>total storey drift</td>
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<td>contribution to the total storey drift from the column, calculated using the slope deflection method</td>
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<td>contribution to the total storey drift from the link</td>
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<td>rotation of the beam-end connection in double channel link subassemblage</td>
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<td>$\theta_{link}'$</td>
<td>link rotation</td>
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<td>$\theta_{StubConn}$</td>
<td>rotation of the stub-end connection in double channel link subassemblage</td>
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<td>Definition</td>
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<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$\theta_p$</td>
<td>total plastic rotation, obtained by removing the elastic portion of $\theta$</td>
</tr>
<tr>
<td>$\theta_{pz}$</td>
<td>contribution to the total storey drift from column panel zone shear deformation</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>regression coefficient for bolt resistance in eccentric bolt group</td>
</tr>
<tr>
<td>$\mu$</td>
<td>regression coefficient for bolt resistance in eccentric bolt group</td>
</tr>
<tr>
<td>$\sum C_f$</td>
<td>sum of factored lateral loads above the storey</td>
</tr>
<tr>
<td>$\sum V_f$</td>
<td>sum of factored axial compressive loads of all columns in the storey</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>true stress</td>
</tr>
<tr>
<td>$\sigma_{noa}$</td>
<td>measured engineering stress</td>
</tr>
<tr>
<td>$\phi$</td>
<td>resistance factor of structural steel, 0.9</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>resistance factor of bolts, 0.8</td>
</tr>
<tr>
<td>$\phi_{br}$</td>
<td>resistance factor of bearing of steel on bolt, 0.67</td>
</tr>
</tbody>
</table>
1.1 SEISMIC DESIGN PHILOSOPHY OF STEEL MOMENT RESISTING FRAMES

To achieve economic designs while providing life-safety to the occupants, modern seismic design principles take advantage of the ductility of buildings and design structures for only a fraction of the expected elastic lateral load. In a steel moment-resisting frame (MRF), plastic hinges develop near the beam-to-column connections during large seismic events, with the ideal deformation mechanism being the one shown in Figure 1.1. These plastic hinges act as ductile fuses, dissipating energy through stable hysteretic behavior while limiting forces transmitted to other components of the structure. To ensure this ductile behavior and to maintain the integrity of the structure, the beam-to-column connections must be able to sustain large inelastic deformations without brittle fractures or significant loss of strength, while the other structural members should remain essentially elastic. As a result, the basic design philosophy for MRFs can be summarized as follows: i) size the fuse to provide the required level of strength to the frame, ii) detail the fuse to provide the required level of ductility, iii) design and detail all other frame members to be stronger than the forces associated with yielding of the fuse and iv) ensure that specified drift requirements are met.

Figure 1.1  Ideal Deformation Mechanism
1.2 CURRENT PRACTICES AND DRAWBACKS

MRFs with traditional welded flange and bolted web connections were believed to be very ductile systems and were extensively used between the 1960s and the early 1990s. This belief was put into question during the Northridge, California earthquake on January 17, 1994. Brittle fractures developed near the connection welds in over 150 buildings (Bruneau et al. 1998), in many cases without any signs of plastic deformation in the beam. As a result of these failures, different schemes were developed to improve the connection performance. A widely accepted solution was the reduced beam section (RBS) connection, which weakens the beam flanges at a short distance away from the column face. This detail forces the plastic hinge to develop within the RBS, thus reducing forces experienced by the connecting structural elements, including the connection welds. A typical example of a RBS connection is shown in Figure 1.2.

![Figure 1.2 Typical radius cut RBS connection](image)

The RBS connection has been extensively tested and was found to provide satisfactory performance under severe cyclic loading, as discussed in further detail in Chapter 2. However, it has several drawbacks which also apply to the other improved connection schemes. Because the yielding fuse is a part of the beam, strength design and drift design of the structure are interlinked. Drift
requirements often control the design in MRF structures. When member sizes are increased to meet drift requirements, the capacity of the yielding fuse also increases. This in turn leads to larger demands on the other parts of the structure, including columns, floor slabs, connections, and foundations, often resulting in over-designed structures and increased over-all costs. In addition, significant damage can result in the beam from repeated inelastic deformation and localized buckling during a design level earthquake. As the cumulative inelastic action of the structure is unknown, it is difficult to assess the extent of damage and the structure's ability to provide adequate level of safety for any subsequent loading. Furthermore, repair of the beam is very difficult, disruptive, and costly.

1.3  MOMENT FRAMES WITH NONLINEAR REPLACEABLE LINKS

The replaceable link concept effectively eliminates the aforementioned drawbacks. Similar to RBS connections, the beam is weakened at a short distance away from the column face, forcing the plastic hinge to form away from the critical section. Instead of reducing the beam flanges, however, replaceable links with smaller flexural capacity are introduced at the locations of expected inelastic action. The other structural elements in the frame are then designed to remain elastic under the forces associated with yielding of the link.

This concept was first proposed by Balut & Gioncu (2003) to facilitate repairs of MRFs. Because the inelastic deformation is concentrated within the link, this concept allows for quick inspection and replacement of damaged links following a major earthquake, significantly minimizing the disruption time of the structure. It also allows for independent control of beam stiffness and required strength, resulting in more efficient structures. Furthermore, it allows welding of critical elements to be done in the shop, considerably improving construction quality and reducing erection time.

Figure 1.3 illustrates a MRF designed according to the replaceable link concept. The shaded portions of the structure represent elements spanning multiple stories that can be fabricated in the shop, shipped, and connected through the nonlinear links on the site.
1.4 **OBJECTIVE AND SCOPE**

The goal of this study was to develop, assess, and validate preliminary guidelines for the design of replaceable nonlinear links for MRFs. To that end, four beam-to-column connections with replaceable links were designed for the ground floor of a five-storey building located in Victoria, British Columbia, following current design provisions for RBS connections (CISC 2005, AISC 2005b, CSA 2001, NRC-IRC 2005). Full-scale experiments of these subassemblages were conducted to ensure that moment connections with replaceable links can provide the necessary strength and ductility equivalent to existing MRF connections. Finite element models were developed to simulate the experimental results, paving the way for future parametric studies. Finally, preliminary design guidelines are suggested to allow designers to easily implement this detail into practice.

This study does not take into account the effects of slabs and other non-structural elements on the frame. However, previous studies have shown that slabs do not cause detrimental effects on the connection and typically increase the plastic rotational capacity of the frame (Tremblay 1997, Jones et al. 2002). In addition, the specimens tested are exterior connections. Interior connections may experience larger demands on the continuity plates and the panel zone, but extensive guidelines exist on how to account for this in the design of MRFs (CISC 2005, AISC 2005b).
1.5 Organization of the Thesis

Chapter 2 provides a survey of developments leading up to the replaceable link concept. The experimental program is presented in Chapter 3 and its results are analyzed and discussed in Chapter 4. The assumptions and results of the finite element models are described in Chapter 5. Preliminary design guidelines are formulated and presented in Chapter 6. Finally, conclusions are drawn in Chapter 7 and recommendations for future research are suggested.
CHAPTER 2: LITERATURE REVIEW

This chapter provides an overview of recent developments in the design of beam-to-column connections in steel moment resisting frames (MRFs) leading up to the replaceable link concept. The historical background is briefly described, followed by a more in-depth discussion of the investigations of the reduced beam section (RBS) connection in recent years. Finally, the theoretical development of the replaceable link concept is presented.

2.1 HISTORICAL BACKGROUND OF MOMENT RESISTING FRAMES

During the early 20th Century, moment frames with heavily riveted connections acquired reputation as excellent seismic resistant systems. As high strength bolts and welding became more common during the 1950s, rivets were replaced by these modern connectors. During early tests of these modern connections, loading protocols and acceptance criteria were vastly inconsistent between different studies. Connection rotation capacity was later adopted as a standard performance indicator, and the acceptable rotation capacity was gradually increased as seismic behavior of steel MRFs became better understood (Civjan, 1998).

The first cyclic tests on moment connections were conducted in the 1960s by Popov and Pinkney (1969), which showed that beam-to-column connections with welded flanges and welded web performed better than cover plate or fully bolted connections. A follow up study by Popov and Stephen (1970) concluded that connections with welded flanges and bolted web also provide adequate seismic performance. In the subsequent years, studies (Krawinkler et al. 1971, Bertero et al. 1973) investigated the effects of panel zone yielding and helped to make the welded flange and bolted web detail a prequalified connection. Code provisions on doubler and continuity plate requirements were developed following further tests by Popov et al. (1986). Popov and Tsai (1989) found that size effects exist in the connection behavior. As a result, supplemental web welds were required for the prequalified connection detail if the beam web contributes to more than 30% of the beam plastic modulus.
Due to their ease of construction, these welded flanges and bolted web connections became widely used in North America. Initially, nearly all bays in a system using ductile moment resisting frames were designed with full moment connections. Overtime, however, many engineers began limiting the number of bays designed as ductile moment frames because full moment connections were more costly than simple shear connections (Bruneau et al. 1998). This practice resulted in a significant loss of structural redundancy, and led to the use of larger beam and column sections with little prior experimental investigation. While conducting further research related to size effects on larger connections, Engelhardt and Husain (1993) found that none of their specimens could provide rotational capacity greater than 0.015 radian, nor could most connections in previous tests conducted by other researchers. Many specimens developed brittle fractures near welds. The two researchers raised concerns about the performance of the welded flanges and bolted web connection detail in severe earthquakes. Their concerns were proven valid by the Northridge Earthquake that immediately followed.

On January 17, 1994, the most costly earthquake in the US history shook the Los Angeles area. The 6.7 magnitude event caused widespread structural and nonstructural damage to buildings and infrastructure. Steel MRFs initially appeared to have survived without significant damage. During the subsequent months, however, over 100 frames were found to be damaged, much to the surprise of the structural engineering community. The vast majority of these damages were brittle fractures at or near beam bottom flanges, often without signs of plastic beam deformation (Youssef et al 1995).

2.2 Post-Northridge Developments

To investigate these damages and to develop solutions, the SAC Joint Venture was formed and was responsible for administering a 5-year research program initiated by the U.S. Federal Emergency Management Agency (FEMA). The FEMA/SAC steel program funded over 50 research projects in six major areas: materials and fracture, joining and inspection, connection performance, system performance, performance prediction and evaluation, and past performance of steel
buildings in earthquakes. Design recommendations and state-of-the-art reports were published in 2000 (Kunnath and Malley, 2002).

Numerous potential causes for the brittle fractures were identified, including inadequate workmanship and inspection quality, poor filler material and welding procedure, over-strength of the beam material, stress concentration caused by the backing bar, and triaxial forces at the column face (Yang et al. 1995, Kaufmann et al. 1997). Two main design strategies that relocate the plastic hinge away from the face of the column were developed to improve connection performance. The first strategy strengthens the connection with cover plates, ribs, side plates, and haunches, while the second strategy weakens the beam at a short distance away from the column face by reducing the area of the beam flanges. Both were experimentally investigated and were shown to provide significantly improved performance over the pre-Northridge connections (Bruneau 1998). Satisfactory performance required that a connection achieve plastic rotations of 0.03 radian without strength degradation of more than 20% of the plastic moment (SAC Interim Guidelines 1995), loaded in compliance with the ATC-24 loading protocol.

Improved welding practices were also recommended and integrated into these new design solutions. Such practices include using filler metal with higher notch toughness, removing backing bars, and using weld access holes to facilitate welding of the bottom flange (Engelhardt et al. 1998, Uang et al. 2000).

2.3 MOMENT CONNECTIONS WITH REDUCED BEAM SECTIONS

While both the strengthening strategy and the weakening strategy lead to satisfactory connection performance, strengthened connections result in increased column sizes, increased demand on the panel zone, and increased rotation demand on the plastic hinge. Weakening a beam by trimming the beam flanges, on the other hand, is an economic solution because it does not require additional material or field work. Demands on the column, the continuity plates, and the panel zone are also decreased because of decreased strength of the plastic hinge. As a result, reduced beam sections (RBS) have gained broad acceptance in a relatively short time (Bruneau 1998). In the years following the Northridge earthquake, a large number of research projects were undertaken to develop this
concept. Their results are summarized here in five sections: i) experimental investigations of beam-to-column connections with RBS, ii) finite element analyses of RBS connections, iii) elastic storey drift increase due to RBS, iv) cyclic stability of RBS connections, and v) analytical models of building frames with RBS. Finally, the current design provisions for RBS connections are presented.

2.3.1 Experimental Investigations of RBS Connections

The concept of RBS was first proposed and tested by Plumier in 1990, for situations such as when delivered steel has significant material overstrength. Research was conducted by ARBED and the concept was patented in the US in 1992. Following the Northridge earthquake, however, commercial royalty rights were waived. Chen and Yeh (1994) also conducted experiments on this concept prior to the Northridge earthquake.

RBS connections were extensively tested following the Northridge earthquake. Three different shapes of cutouts were investigated: constant cut, tapered cut, and radius cut. Constant cut specimens were tested by Plumier (1997) and Engelhardt (1996). While Plumier found that constant cut RBS prevents cracking in the beam near the weld, the specimen tested by Engelhardt experienced flange fracture in the reduced beam section. Tapered cut RBS has linearly varying cutouts intended to approximately follow the moment gradient. The majority of the fifteen specimens tested by Iwankiw and Carter (1996), Chen et al. (1996) and Zekioglu et al. (1997) reached 0.03 radian plastic rotation or above, however a number of them also experienced fractures at the narrowest section of the beam flange. For improved ductility, smooth transition is needed around the sharp corners for these two configurations. Because this additional procedure is needed and because the cuts are sensitive to fabrication flaws, RBS with constant and tapered cuts are not commonly used. Radius cut RBS (shown in Figure 2.1) showed excellent performance in early tests by Engelhardt et al. (1996, 1998) and Popov (1998). All five specimens experienced gradual strength deterioration due to local and lateral buckling, and no fractures were observed. In addition, cost comparisons by W&W Steel showed that the radius cut RBS connection was the least costly among nine different connection types considered for a 25 storey steel office building in Salt Lake City (Engelhardt 1996). As a result, almost all additional RBS research focused on the radius cut RBS.
During phase II of the SAC steel project and beyond, researchers further investigated how different factors influence RBS behavior.

- The effect of composite concrete slab on RBS connection was studied by Tremblay et al. (1997) and Jones et al. (2002). It was found that the presence of a slab did not promote early fracture. On the contrary, the slab enhances beam stability and delays strength degradation. Jones et al. recommended against the use of shear studs and other local stress risers within the RBS region.
- Weak-axis RBS connections were tested by Gilton and Uang (2002). Both specimens reached 0.03 radian plastic rotation without brittle fractures or significant strength degradation. Design recommendations were presented.
- RBS connections with deep columns were investigated by Chi and Uang (2002) and Zhang and Ricles (2006). Chi and Uang found that the deep column is prone to twisting, caused by eccentric beam flange forces due to lateral-torsional buckling of the beam. The design recommendations they formulated were improved by Zhang and Ricles (2006), who found that composite slab and lateral bracing reduce the torque experienced by the deep column.
- Because RBS reduces the forces experienced by the beam and column, Pantelides et al. (2004) investigated the performance of RBS connections without continuity plates as specified by the current design provisions and found that all four test specimens performed well. Adan (2006) continued this investigation and provided guidelines to determine the need for continuity plates for column sections up to W30x191.
• Noting that the majority of previously tested RBS specimens used fully welded web connections, Lee and Kim (2007) investigated the response of RBS connection with bolted web and found it to be more vulnerable to weld fracture. They proposed and validated an improved bolted web configuration according to actual load transfer mechanism from the beam web to the column flange.

RBS has also been incorporated into different lateral load resisting systems with success, including moment connections to concrete-filled circular hollow section columns (Wang et al. 2007) and steel plate shear walls (Qu et al. 2008)

2.3.2 Finite Element Analyses of RBS Connections

In addition to experimental investigations, a few of the experiments presented above were accompanied by finite element analyses to further understand the behavior of RBS connections (Zekioglu et al. 1997, Jones 2000, Gilton and Uang 2002, Zhang and Ricles 2006, Adan 2006, Lee and Kim 2007). The analyses were conducted using commercial software ANSYS or ABAQUS. The finite element models were first calibrated using experimental results. The calibrated models were then used to perform parametric studies which in turn helped to formulate design recommendations. The majority of these models used four node quadrilateral shell elements with reduced integration and large-strain formulation. As computing platforms became more powerful, more computationally intensive models also became possible. Zhang and Ricles (2006) employed detailed submodels (including welds and weld access holes) with eight node solid elements in the region of interest to obtain more accurate stress-strain results. Lee and Kim (2007) used eight node solid elements for the entire global model and also incorporated contact interaction between the shear tab and the beam web for the bolted web connection. Bolts were not included in the model and bolt pretension forces were approximated using point loads.

To achieve the correct buckling shape at the RBS location, linear eigenvalue buckling analysis is first performed. A small fraction of the first buckling mode is then imposed on the specimen as initial imperfection prior to the nonlinear analysis. Models by Jones (2000) as well as Gilton and Uang (2002) were only loaded monotonically, whereas models from the other studies also underwent cyclic loading.
Crack propagation was not addressed in these studies. Rather, the potential for cracking was investigated through the analysis of stress and strain at critical locations.

### 2.3.3 Cyclic Stability of RBS Connections

The majority of RBS specimens tested experienced strength degradation due to buckling. As a result, research has also been conducted to study the stability of RBS connections, especially under cyclic loading.

Uang and Fan (2001) performed regression analysis based on the test results of 55 full-scale RBS specimens. It was observed that plastic rotation capacity and strength degradation rate are highly dependent on the slenderness ratio of web local buckling. In order to reach 0.03 radian plastic rotation, the limit for web slenderness ratio $\frac{h}{w}$ is suggested to be $1100 / (\sqrt{F_y})$ (with $F_y$ in MPa), which is more conservative than that stipulated in the AISC Seismic Provisions (1997). The presence of a concrete slab was found to increase the plastic rotation capacity of a RBS connection under positive bending only.

Nakashima et al. (2002) performed numerical analyses to study the lateral-torsional instability and lateral bracing effects, using a program written by the authors. The numerical model was validated using reduced-scale steel beam tests. The study confirmed that the unbraced length requirement in the AISC Seismic Provisions (1997) is reasonably conservative to ensure sufficient rotation capacity. It was also noted that the RBS beam is not necessarily more susceptible to lateral instability than the corresponding standard beam. The study did not consider local buckling, however, and caution was urged in using these results.

### 2.3.4 Elastic Storey Drift Evaluation of Moment Frames with RBS

In addition to studies of local behavior at the RBS connections, a number of analytical studies were also conducted to investigate the effects of RBS on the seismic response of the entire moment frame. Reducing the flange area at the RBS location leads to decreased beam stiffness, which in turn reduces the stiffness of the entire moment frame. An accurate estimate of this stiffness loss is
desirable to evaluate whether the structure meets the drift requirements. Numerous studies have been undertaken to develop both exact and approximate solutions to this problem.

Grubbs (1997) performed elastic finite element analyses of three different sizes of beams to examine the effect of radius cut RBS on beam stiffness. A simplified approach was developed, allowing the circular reduced region to be modelled as a single prismatic frame element. Linear regression analysis was used to formulate an equation for the effective moment of inertia of the RBS region. Frame analyses were then performed on several moment frames, using the simplified approximation for the RBS location. It was found that 40% and 50% flange reduction correspond to 4%-5% and 5%-7% reduction in frame stiffness, respectively.

Noting the limited number of beam sizes studied by Grubbs, Jin (2002) estimated the effective moment of inertia of circular RBS regions through regression analysis of 22 beam sections. Using this estimate, stiffness reduction in the beam and drift increase of the frame were further derived using the method of virtual work. Elastic analysis performed for three steel moment frames indicated that the stiffness modification factor of 9% suggested in FEMA 350 (2000) is conservative, especially for shorter buildings.

Iwankiw (2002) theoretically derived a stiffness formulation of a beam with circular RBS using the conjugate beam method. Using the derived formulae, parametric analysis was performed for 40 beams that cover design ranges of practical interest. Approximate linear regression equations were provided as an alternative to the lengthy derivation, yielding reasonable results. The length of the reduced beam section was found to have a dominant effect on frame stiffness reduction.

Chambers et al. (2003) independently derived a stiffness matrix for beams with circular RBS based on a closed-form virtual work solution. The derived matrix was validated using finite element analyses of a cantilever beam. A parametric study was performed on beams with different RBS configurations as well as on two different building frames, within the range of practical interest. Positive correlation was observed between increased storey drift and i) RBS length, ii) percent reduction, and iii) distance between RBS and the column face. In one extreme case, 10% storey drift increase was observed for 40% flange reduction when the RBS is very close to the column face.
To simplify the evaluation process, Lee and Chung (2006) took an approach similar to Jin (2002) and replaced the radius-cut RBS with an equivalent segment of constant width. A mathematical derivation was used instead of a linear regression, and the results were validated using elastic finite element analyses. The conjugate beam method was then used to derive equations that can be used to estimate the expected storey drift increase. Case studies were performed within the range of practical interest, and drift increases on the order of 4-9% were observed.

### 2.3.5 Performance Evaluation of Building Frames with RBS

While the aforementioned studies focused on the elastic stiffness of moment frames with RBS connection, several studies were also investigated the inelastic performance of RBS moment frames. Jones (2002) developed a constitutive model to predict the load-displacement relationship of a RBS subassemblage, which isolates the drift contributions of the different components (RBS, panel zone, elastic portions of beam and column). The model was verified analytically using finite element analysis and experimentally using full-scale specimens. It was then used in nonlinear dynamic analysis of a 13 storey building in Southern California. Jones (2002) suggested that the model can be used in the performance based design of a symmetric RBS moment frame.

Lee and Foutch (2002) proposed a new procedure that determines the confidence level for a moment frame to achieve a given performance objective. The researchers designed 3-, 9-, and 20-storey buildings with RBS and developed analytical models that also account for the contributions of interior gravity frames. Static pushover analyses were performed. Drift capacity and demands were evaluated using incremental dynamic analysis. Very high confidence levels were obtained for the frames for both the collapse prevention and immediate occupancy performance levels.

Using the procedure proposed by Lee and Foutch (2002), Jin and El-Tawil (2004) performed nonlinear pushover and transient analyses of 4-, 8-, and 16-storey RBS frames in the Los Angeles area. A suite of 20 scaled earthquakes were used for each building and for two hazard levels: immediate occupancy and collapse prevention. The study confirmed that frames with RBS can provide good seismic performance in regions of high seismic risk. The researchers also observed
that the pushover procedure as specified in FEMA 350 (2000) can substantially overestimate demands.

2.3.6 Current Design Provisions for RBS

Based on the test results of 27 radius cut RBS specimens, Moore et al. (1999) provided design recommendations for fully restrained, radius cut RBS connections. A step-by-step procedure is provided, with commentary for various design considerations. A very similar procedure is included in FEMA-350 (2000) and AISC (2005b), which also provides design guidelines for other prequalified post-Northridge connections. These provisions were adapted for Canadian design practice in CISC (2005). In this code document, RBS connections are pre-qualified for columns sizes smaller than W360.

2.4 Replaceable Link Concept

The large number of research projects on RBS connections increased the confidence level in this particular detail. These investigations have shown that the reduced beam section is indeed an efficient way to improve the ductility of steel moment frames. However, it is very difficult to repair a beam after it has been damaged through repeated inelastic deformation, as Balut and Gioncu (2003) pointed out. Instead of trimming away the beam flanges, they proposed using dismountable dissipative elements which can be removed and replaced, while the rest of the structure remains linear elastic. Two variations of this dismountable RBS were envisioned: a W-section with end plates or back-to-back channels eccentrically bolted to the beam web. The second variation was thought to be preferable because it provides a more gradual transmission of forces at the connections through friction. For increased local stability, it was suggested that channel webs be connected using bolts and a packing plate in the interval between the two beam segments.
2.4.1 \textit{W-Section with End Plate Connection}

End plate connections have been previously used to directly connect beams to column flanges. The beam is shop welded to a thick end plate using complete joint penetration welds. The end plate is then field bolted to the column flange using prestressed high strength bolts.

The end plates may be flush with or extend beyond the beam flanges. Extended end plate connections were investigated as alternatives to welded connections during the aftermath of the Northridge Earthquake, and two variations were chosen as prequalified connections: the four bolt unstiffened end plate connection and the eight bolt stiffened end plate connection. Design procedures for these two connections subjected to cyclic loading were developed by Sumner and Murray (2002) and were included in FEMA 350 (2000). Sumner (2003) also experimentally and analytically investigated the behaviour of six additional end-plate moment configurations and proposed corresponding unified design procedures. These procedures were based on yield-line analyses for end-plate design and the modified Kennedy method for bolt design (Sumner 2003).

2.4.2 \textit{Double Channels with Eccentrically Bolted Connection}

A limited number of studies have been conducted to investigate the behavior of double channel built-up sections under reversed cyclic bending. Parra-Montesinos et al. (2006) performed experiments on six cantilever specimens to evaluate the stitching space and lateral bracing space required to sustain large inelastic rotations. These double channel built sections were intended as truss girders in special truss moment frames, and were connected using welded reinforcing gusset plates. The study found that current stitching spacing and lateral bracing requirements given in the AISC LRFD provisions are not adequate to ensure large rotation capacity in these built up members. A new equation was proposed, which decreases the current limit by approximately 50%. Lateral bracing was deemed necessary in the region adjacent to the plastic hinge to achieve 0.06 radian total plastic rotation of the hinge. Good rotational capacity was observed for the specimens that met these new criteria, although large over-strength was also observed in these built-up channel sections.
Eccentrically bolted connections are commonly analyzed using the method of instantaneous center of rotation, as described by Kulak et al. (1987). This method is applicable for both bearing-type and slip-critical connections.

The goal of this study is to design and validate moment connections with both variations of replaceable links, in consultation with relevant literature as summarized in this Chapter.
CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 Objectives

Four full scale tests of exterior beam-to-column subassemblages were carried out at the University of Toronto to evaluate the replaceable link concept for MRFs. The main objective of these tests was to verify that frames with replaceable links can reach the required plastic rotation without significant damage and without deterioration of their load carrying capacity. Furthermore, the experiments were also designed to verify the replaceability of these links and to observe issues related to the replacement operation.

Two different link concepts were tested: i) with welded end plates and ii) with back-to-back double channels. Results for the two concepts were compared and the effectiveness of each concept was evaluated. Connection performance for the double channel links were studied, especially the effects of bolt slippage on the overall performance of the link.

Few experiments have been conducted on the cyclic inelastic behaviour of double channels sections yielding in flexure. This experimental program also aimed to provide a better understanding of their behaviour.

3.2 Test Subassemblage

The test set-up simulates a typical first floor exterior beam-to-column joint of a five storey building located in Victoria, BC. The assumed inflection points are at the center of the beam, the center of the column above, and 0.7hs from the ground on the column below to account for the fixity of the column at the base. The subassemblage includes the beam-to-column connection as well as the beam and column extending to the inflection points. This set-up has been used widely to test beam-to-column connections in moment resisting frames, including studies conducted by Iwankiw and Carter (1996), Engelhardt et al. (1998), and Chi and Uang (2002).

A schematic drawing of the test set-up is shown in Figure 3.1(a). The inflection points in the columns are idealized as pinned-connections. Vertical load is applied at the beam inflection point. The resulting subassemblage is statically determinate and experiences constant shear and single
curvature moment in the beam. The general bending moments and shear forces are shown in Figures 3.1(b) and 3.1(c), respectively.

![Diagram of deflected shape, bending moment, and shear force](image)

**Figure 3.1** (a) Deflected shape, (b) bending moment, (c) shear force

This set-up configuration was connected to the strong wall and strong floor at the University of Toronto laboratories. The column is attached to the strong wall reaction frame using two \(2\frac{3}{4}\) inch pins. The base of the column is attached to a short segment of a W250x101 section to achieve a near-pinned boundary condition. Interstorey drift is imposed on the beam by a 2650kN Mobile Testing Machine (MTS), 3.3 metres away from the column face. The detailed test set-up is shown in Figure 3.2. The W250x101 section is held down by its bottom flange, using two HSS203x203x13 sections post-tensioned to the strong floor (not shown).

Axial loads were not imposed on the column and the composite slab is not included in the experiments. The amount of axial load usually experienced by columns in the field has little influence on the beam-to-column connection response (Whittaker et al. 2000). The composite slab helps to delay flange and web buckling in the field, thus preventing beam shortening (Tremblay et al. 1997). The absence of the slab during the experiment is therefore expected to represent a worst-case scenario performance.

To prevent lateral-torsional buckling of the subassemblage, lateral bracing columns are placed at a distance of 0.88m away from the far end of the link. Due to limitations of the strong floor bolt pattern, this distance is longer than the recommended distance of \(d/2\) and therefore likely represents a less favourable bracing configuration. The lateral bracing columns are able to resist 6%
of the axial strength of the link flange, as required by the AISC Seismic Provisions for Structural Steel Buildings (2005).

Lateral supports are also provided near the point of loading as well as the pinned reactions of the column. The pins used do not restrict out of plane movements, thus the lateral supports are needed to maintain overall stability of the test fixture.

![Figure 3.2 Set-up elevation view](image-url)
3.3 Test Specimens

This section provides a general description of the test specimens. A detailed review of specimen design is presented in Appendix B.

3.3.1 Links

For each link type, two test specimens with nearly identical geometric and material properties were fabricated. The duplicate specimens were necessary to assess the replaceability of the link and the performance of the replacement link in comparison to the original link. Different surface conditions were provided to investigate the effect of different friction capacity on the connection performance. Table 3-1 summarizes the specimen test matrix.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Link Section</th>
<th>Connection Type</th>
<th>Surface Condition</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF-1A</td>
<td>W460x128</td>
<td>Eccentrically Bolted Web</td>
<td>Mill Scale (A)</td>
<td>3/8&quot; web doubler plate at connection</td>
</tr>
<tr>
<td>MRF-1B</td>
<td>W460x89</td>
<td>Bolted End Plate</td>
<td>Blast-cleaned (B)</td>
<td>1/4&quot; web doubler plate at connection</td>
</tr>
<tr>
<td>MRF-2A</td>
<td>cut W460x89</td>
<td>Bolted End Plate</td>
<td>Mill Scale (A)</td>
<td>n/a</td>
</tr>
<tr>
<td>MRF-2A</td>
<td>W460x89</td>
<td>Blast-cleaned (B)</td>
<td>Mill Scale (A)</td>
<td>n/a</td>
</tr>
</tbody>
</table>

3.3.1.1 Specimens 1A and 1B

Specimens 1A and 1B are double channel links manufactured from W460x89 class 1 sections. The W-sections are used to obtain thinner flanges and thicker webs for a given section depth, thereby ensuring that flexural yielding occurs before shear yielding. To form the channels, the flanges on one side of the W-section are cut flush with the web, thus retaining the full web thickness. End stiffeners are added to the the channels to constrain the movements of the the top and bottom flanges.

To prevent buckling of the individual channels, the back-to-back channels are connected to each other with 5/8" bolts that fasten the link webs, preventing the webs from buckling outwards.
Packing plates with a total thickness of 38mm, equal to the thickness of the beam web plus its doubler plates, are inserted between the two channels to prevent inward buckling. Because the flanges of these flexural links experience the highest stresses, they are kept free to prevent undesirable stress concentrations.

The double channel links are connected to the beam and beam-stub by eccentrically bolted connections. All bolts used are 1" A490 bolts. To minimize bolt hole deformation and hence improve the connection performance of the links, doubler plates are fillet welded to the connection region. To assess the bearing behavior of bolt groups under large cyclic eccentric loads, the two specimens had doubler plates of different thickness: 1/4" for specimen 1B and 3/8" for specimen 1A. Figure 3.3 shows important details of the double channel links.

3.3.1.2 Specimens 2A & 2B

Specimens 2A and 2B were fabricated from W460x128 class 1 sections. End plates with length and width equal to the outer beam dimensions are connected to the W sections using complete joint penetration welds. These details are based on recommendations for welded beam-to-column connections. Welding was performed from the underside of the flanges, through the weld access holes. In practice, however, welding should be performed from the other side of the flanges to allow continuous weld passes.

The links are connected to the beam and beam-stubs using bolted end plate connections. All bolts used are 1" A490 bolts. Figure 3.4 shows important details of the end-plate links.

3.3.2 Connecting Beam and Column

The same beam and column were used for all four tests. Each end of the beam is designed to connect to one type of link and the column has two welded beam stubs. After the tests are completed for one type of link, the column and beam are inverted for tests of the other link type. Figures 3.5 and 3.6 show the beam and column details.

The beam and column were designed according to CAN/CSA-S16-01(CSA 2001) and Moment Connections for Seismic Application design guide (CISC 2005). Both elements, including the panel
zone, were capacity designed to remain elastic through all four tests. The beam stubs were welded to the column following recommendations for welded beam-to-column connections.

Figure 3.3  MRF-1 Link Details

- All holes are 1 1/16” UON
- Doubler plates: 3/8” on MRF-1A
  1/4” on MRF-1B
Figure 3.4  MRF-2 Link Geometry
Figure 3.5  Beam-column connection details for MRF-1
Figure 3.6  Beam-column connection details for MRF-2
3.3.3 Material Properties

The rolled steel sections used to fabricate the links, beam, and column are CSA G40.21 350W grade steel, with nominal \( F_y = 350 \) MPa.

Because the beam and column were both expected to remain linear elastic, coupon tests were conducted only for the link material. Standard plate type tensile coupons were cut from stub sections that were produced from the same heat as the link specimens. The measured yield and ultimate stress and strain values are summarized in Table 3-2. The measured yield stress for flanges of MRF-2 specimens is slightly higher than the \( R_y, F_y \) allowed by CAN/CSA S16-01.

<table>
<thead>
<tr>
<th>Section</th>
<th>Coupon Location</th>
<th>( F_y ) (MPa)</th>
<th>( F_u ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W460x89</td>
<td>Flange</td>
<td>350</td>
<td>484</td>
</tr>
<tr>
<td>(MRF-1)</td>
<td>Web</td>
<td>370</td>
<td>481</td>
</tr>
<tr>
<td>W460x128</td>
<td>Flange</td>
<td>390</td>
<td>499</td>
</tr>
<tr>
<td>(MRF-2)</td>
<td>Web</td>
<td>442</td>
<td>538</td>
</tr>
</tbody>
</table>

3.4 Fabrication

All test specimens were fabricated by Walters Inc. of Hamilton, Ontario. Doubler plates for the double channel connections and the panel zones were prepared and welded at the University of Toronto laboratories. All welding was performed according to the Flux Cored Arc Welding (FCAW) process using E49XX grade electrodes.

In practice, the links and surrounding components are intended to be welded in the shop and bolted on site, thus there was no need to simulate field welding conditions and no special welding positions were required for the fabrication. Welding inspections were also not required.
3.5 LOADING PROTOCOL

The loading protocol specified in Appendix S6.2 of the AISC Seismic Provisions for Structural Steel Buildings (2005) was used for the testing program (Table 3-3). This cyclic displacement history consisted of symmetric, stepwise-increasing displacements that were imposed on the beam by the MTS loading frame, at a distance of 3.3m away from the column face. Loading was continued until the test set-up deflection limit was reached. The specimens were then cycled at the largest possible rotation until a severe fracture is observed or until the strength deteriorated below 60% of the peak load.

TABLE 3-3  Loading Sequence

<table>
<thead>
<tr>
<th>Total storey drift angle (rad)</th>
<th>Number of loading cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00375</td>
<td>6</td>
</tr>
<tr>
<td>0.005</td>
<td>6</td>
</tr>
<tr>
<td>0.0075</td>
<td>6</td>
</tr>
<tr>
<td>0.01</td>
<td>4</td>
</tr>
<tr>
<td>0.015</td>
<td>2</td>
</tr>
<tr>
<td>0.02</td>
<td>2</td>
</tr>
<tr>
<td>0.03</td>
<td>2</td>
</tr>
<tr>
<td>0.04</td>
<td>2</td>
</tr>
</tbody>
</table>

Additional cycles continue with 2 cycles for each 0.01 radian increment.

3.6 ACCEPTANCE CRITERIA

The CAN/CSA S16-01 seismic provisions state that in order for a reduced beam section design to qualify for use, the beam needs to undergo the loading protocol outlined in Section 3.5 and sustain a moment at the column face above or equal to $0.8M_{p, beam}$ for one complete cycle at 0.04 radian storey drift. This acceptance criteria cannot be applied to a subassemblage with a replaceable link, however. The design of the link is de-coupled from the design of the beam, thus its strength may be much smaller than that of the smallest possible RBS for the beam. Since the moment at the column face is dependent on the probable strength of the link, it may never reach $0.8M_{p, beam}$ in a replaceable link specimen. Thus an acceptance criteria from earlier MRF studies (Engelhardt et al. 2002, Uang et al. 2002) is used for this test program: the subassemblage must follow the specified
loading protocol and complete one cycle at 0.04 radian storey drift without strength deterioration below 80% of the peak load achieved during the test.

### 3.7 Instrumentation

The global instrumentation scheme included:

- A displacement transducer along the MTS centerline, measuring the imposed displacement
- A load cell along the MTS centerline, measuring the applied load
- Displacement transducers at column-end connections, measuring the rigid body displacement of the column

The test was controlled by the imposed displacement minus the displacement due to the rigid body rotation of the column.

Additional displacement transducers were used to measure the panel zone deformations and link rotations. The holders for these transducers were tack-welded to the beam and the column. Uniaxial and rosette strain gauges were strategically placed to measure strains in webs, flanges, and stiffeners of the links and connecting regions. The data from the instrumentation was acquired at a rate of 2 Hz with the HBM MGCplus data acquisition system and version 4.5 of the Catman software.

Displacement transducer locations are shown in Figures 3.7 and 3.8. Table 3-4 lists the types and ranges of displacement transducers used for each test. Strain gauge locations are shown in Figure 3.9.
<table>
<thead>
<tr>
<th>Displacement Transducers</th>
<th>MRF-1A</th>
<th>MRF-1B</th>
<th>MRF-2A</th>
<th>MRF-2B</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>+/- 10&quot; String Pot</td>
<td>+/- 10&quot; String Pot</td>
<td>+/- 5&quot; LVDT</td>
<td>+/- 10&quot; String Pot</td>
</tr>
<tr>
<td>D2</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 1&quot; LVDT</td>
</tr>
<tr>
<td>D3</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
</tr>
<tr>
<td>D4</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
</tr>
<tr>
<td>D5</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
<td>+/- 1/2&quot; LVDT</td>
</tr>
<tr>
<td>D6</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 1&quot; LVDT</td>
</tr>
<tr>
<td>D7</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 1&quot; LVDT</td>
</tr>
<tr>
<td>D8</td>
<td>+/- 3/4&quot; Pot</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>D9</td>
<td>+/- 3/4&quot; Pot</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>D10</td>
<td>+/- 3/4&quot; Pot</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>D11</td>
<td>+/- 3/4&quot; Pot</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>D12</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 2&quot; Pot</td>
</tr>
<tr>
<td>D13</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 2&quot; Pot</td>
<td>+/- 1&quot; LVDT</td>
<td>+/- 2&quot; Pot</td>
</tr>
</tbody>
</table>

Note: LVDT = linearly variable differential transducer, Pot = potentiometer
Figure 3.7 MRF-1 instrumentation

Displacement Transducers:
- D1: Vertical displacement at load point
- D2: Relative vertical displacement of link ends
- D3, D4: Horizontal displacement of column ends
- D5: Vertical displacement at column base
- D6, D7: Diagonal panel zone deformation
- D8, D9, D10, D11: Connection movement (MRF-1A only)
- D12, D13: Link deformation
Figure 3.8  MRF-2 instrumentation

Displacement Transducers:

D1: Vertical displacement at load point
D2: Relative vertical displacement of link ends
D3, D4: Horizontal displacement of column ends
D5: Vertical displacement at column base
D6, D7: Diagonal panel zone deformation
D12, D13: Link deformation
Figure 3.9  (a)MRF-1 strain gauge, (b)MRF-2 strain gauge
CHAPTER 4: EXPERIMENTAL RESULTS

4.1 INTRODUCTION

This chapter presents results from the experimental program. For each specimen, the results are first described in three parts: experimental observations, global hysteretic behavior, and local responses. Performance of the different specimens are then compared based on their energy dissipation capacity. Finally, the installation and replacement processes are discussed.

The specimens are presented in the order in which they were tested. Each test was conducted over the course of several days. At key points during the experiments, loading was paused for visual inspection and photography. Table 4-1 tabulates the test date, the maximum storey drift (\( \theta \)) and total plastic rotation (\( \theta_p \)) achieved in the last complete cycle before strength deteriorated below 80% of peak load, and mode of failure for each specimen. Figure 4.1 shows the four replaceable links after removal from the test set-up.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Date</th>
<th>( \theta ) *</th>
<th>( \theta_p )*</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF-2A</td>
<td>Oct 30-Nov 1</td>
<td>0.04</td>
<td>0.031</td>
<td>Strength deteriorated below 80% of peak load due to local flange and web buckling</td>
</tr>
<tr>
<td>MRF-2B</td>
<td>Nov 7 - 8</td>
<td>0.04</td>
<td>0.031</td>
<td>Strength deteriorated below 80% of peak load due to local flange and web buckling</td>
</tr>
<tr>
<td>MRF-1B</td>
<td>Nov 16 - 20</td>
<td>0.06</td>
<td>0.049</td>
<td>Strength deteriorated below 80% of peak load due to brittle fractures in flanges and web</td>
</tr>
<tr>
<td>MRF-1A</td>
<td>Nov 23 - 28</td>
<td>0.07</td>
<td>0.059</td>
<td>Strength deteriorated below 80% of peak load due to slowly propagating cracks in flanges and webs</td>
</tr>
</tbody>
</table>

*Drift and rotation reached in the last complete cycle before strength deteriorated below 80% of peak load

4.2 PRESENTATION OF RESULTS

The links experience asymmetrical loading along their length because of the moment gradient. To distinguish between the different sides of the links while discussing the results, the side connected to the beam-stub is referred to as the stub side, while the side connected to the beam is referred to as the beam side. For the double channel links, the channel with whitewash is referred to
as the whitewashed channel, while the channel with more strain gauges is referred to as the instrumentation channel.

The experimental observations described for each specimen include initiation and propagation of yielding, buckling and cracking, as well as the onset of bolt slippage. A figure is presented for each specimen, summarizing the observed occurrence of these events during the course of loading. When referring to a specific cycle in the loading sequence, the following notation is used: storey drift angle, cycle number, and positive or negative excursion. For example, 0.0075-1P refers to the positive excursion of the first cycle at a storey drift of 0.0075 radians. Positive values of loads and displacements corresponds to upward movement. Although visual inspections were performed frequently, these events may have taken place earlier than observed. Photographs of each specimen during various stages of loading, as well as close-ups of critical details, are presented. The specimens
were whitewashed on one side to clearly show the extent of yielding, thus the majority of the photos were taken on that side.

For each specimen, the global hysteretic response is summarized by three figures. The first figure shows the applied load on the beam, \( F_b \), against the imposed displacement on the beam, \( \delta_b \). The storey drift angle, \( \theta \), and the moment at the column face, \( M_{cf} \), are also labelled on the same plot. The value of \( \theta \) is calculated according to Equation (4.1), where \( L \) (3.51m) is the distance from the point of loading to the column centerline. The moment at the column face is calculated according to Equation (4.2), where \( d_c \) (0.416m) is the column section depth. Dashed lines indicate the load corresponding to 80% of the peak load experienced during the test.

The second figure shows the hysteretic curve of \( M_{cf} \) vs. \( \theta_p \). The total plastic rotation \( \theta_p \) is obtained by removing the elastic component of the storey drift using Equation (4.3), where \( k_{elastic} \) is the linear elastic stiffness of the subassemblage calculated from the experimental results. Experimental values of \( M_{cf} \) are compared with \( M_{cf} \) values corresponding to the design strength \((0.9M_{p,link})\) and the probable strength of the link \((C_{pr}R_yM_{p,link})\). Detailed calculations of these two link strengths are presented in Appendix B.

\[
\theta = \frac{\delta_b}{L} \tag{4.1}
\]

\[
M_{cf} = F_b \times \left( L \angle \frac{d_c}{2} \right) \tag{4.2}
\]

\[
\theta_p = \theta \angle \frac{F_b}{k_{elastic} \times L} \tag{4.3}
\]

On the third figure, \( \theta \) is further decomposed into contributions from the beam, the column, the panel zone, and the link. Because the beam and the column remained essentially linear elastic during the experiments, their contributions to the total storey drift are computed theoretically using the slope deflection method. The displacement transducers used to calculate panel zone and link contributions for specimens 1A and 1B are shown in Figure 4.2. The position of transducers \( D6 \), \( D7 \), \( D12 \), and \( D13 \) are similarly located on specimens 2A and 2B.
The column panel zone contribution due to shear deformation, $\theta_{pz}$, is calculated using measurements from the displacement transducers $D6$ and $D7$ according to Equation (4.4). The variables $a$ and $b$ are the vertical and horizontal distance between the ends of the transducers, respectively. For tests 2A and 2B, $D6$ and $D7$ were anchored on the edge of the column flanges adjacent to the panel zone. To measure panel zone activities more accurately, the anchors for these two transducers were moved to the column web for tests 1B and 1A.

$$\theta_{pz} = \frac{\sqrt{a^2 + b^2} \times (D7 \angle D6)}{2ab}$$

(4.4)

Drift contribution from the link is obtained in two steps. The link rotation, $\theta'_{link}$, is first calculated according to Equation (4.5) using measurements from displacement transducers $D12$ and $D13$. $h$ is the vertical distance between the two transducers. Because the storey drifts are calculated with respect to the column centerline, $\theta'_{link}$ is then translated into the link contribution to total storey drift ($\theta_{link}$) using Equation (4.6), where $L'$ is the distance from the plastic hinge center to the point of loading. For the end plate links, the plastic hinge center is at the center of the link. For the double channel links, the equivalent center of plastic hinge is shifted approximately 100mm towards the stub side because of the differential connection rotation at the two ends of the link. Because the sum of the four contributions agree well with the measured storey drift, in cases where direct
measurements of link contribution are missing due to problems with the displacement transducers, \( \theta_{\text{link}} \) is calculated using Equation (4.7). The final values presented are averages of values from the first positive and negative excursions at each drift level.

Small displacement potentiometers (D8, D9, D10 and D11) were placed between the beam flanges and the channel flanges of specimen 1A to measure connection rotation at the two ends of the link. Connection rotation on the stub side (\( \theta_{\text{StubConn}}' \)) and the beam side (\( \theta_{\text{BeamConn}}' \)) are calculated according to Equations (4.8) and (4.9), where \( d \) is the horizontal distance between the displacement potentiometers. The sum of \( \theta_{\text{StubConn}}' \) and \( \theta_{\text{BeamConn}}' \) is the total connection contribution to link rotation \( \theta_{\text{link}}' \).

\[
\theta_{\text{link}}' = \frac{D12 \angle D13}{h} \quad (4.5)
\]

\[
\theta_{\text{link}} = \theta_{\text{link}}' \times \frac{L'}{L} \quad (4.6)
\]

\[
\theta_{\text{link}} = \theta \times \theta_{\text{beam}} \times \theta_{\text{column}} \times \theta_{p} \quad (4.7)
\]

\[
\theta_{\text{StubConn}}' = \frac{D9 \angle D8}{d} \quad (4.8)
\]

\[
\theta_{\text{BeamConn}}' = \frac{D10 \angle D11}{d} \quad (4.9)
\]

The local responses of the specimen are studied using data gathered by the strain gauges. The strain profiles across the link flanges and along the length of the link are presented at selected drift levels. The strain in selected stiffeners are also examined to investigate their contributions to specimen performance. All strains are normalized by the strain value at yield (0.002 m/m).
4.3 Specimens MRF-2A and MRF-2B

Specimens 2A and 2B are geometrically identical but have different surface finishing. Specimen 2A had a mill scale finish while specimen 2B was blast-cleaned. The blast-cleaned surface provides a higher coefficient of friction. However, because bolts in the end plate connections are predominately loaded in tension, this difference in friction coefficient was not expected to affect the performance of the two specimens significantly. The connection bolts in specimen 2A were pretensioned by a laboratory technician using a 1-meter long wrench, whereas the bolts in specimen 2B were pretensioned to 70% of their minimum tensile strength using an air wrench.

Test 2A followed the loading protocol up to 0.05 radians and completed 1 cycle at this drift level. Because of axial shortening of the link and second order effects, side-loading on the MTS loading head exceeded the acceptable range at this drift level, therefore loading did not continue following the protocol. The specimen completed two more cycles at 0.04 radian storey drift without further strength degradation, and at that point the test was terminated.

To decrease side-loading on the MTS head, the distance between the two hinges under the loading head was increased for subsequent tests. Test 2B was able to follow the loading protocol up to 0.05 radian without exceeding the acceptable side-loading limit, however it was not permissible to continue to 0.06 radian positive storey drift. The specimen completed 4 cycles at 0.05 radian without significant strength degradation, thus the test was terminated after reaching 0.06 radian at the last negative excursion.

Axial shortening of the links due to flange and web buckling was larger than expected, as a result the displacement transducers \( D_{12} \) and \( D_{13} \) went out of range (±1”) in the higher cycles of test 2A. Transducers with longer gauge length (±2”) were used for test 2B. On the other hand, the transducers used in test 2A to measure panel zone deformation, \( D_6 \) and \( D_7 \), had very long gauge length (±2”) in comparison to the measured displacement. This contrast resulted in measurements with low resolution and less accuracy. Therefore transducers with shorter gauge length (±1”) were used for test 2B.
4.3.1 Specimen MRF-2A

Experimental Observations

A summary of significant events is presented in Figure 4.3. The deformed subassemblage at 0.05 radian storey drift is shown in Figure 4.4.

Yielding in the link was first observed during the 0.01 radian cycles. Whitewash on the link flanges began flaking near the end plates (more pronounced on the stub side than the beam side), then propagated toward the center of the link. During the 0.015 radian cycles, slight gap opening was observed between the stub side end plates near the top link flanges. Yielding propagated into the link webs at 0.02 radian storey drift. Flange buckling was first observed at 0.03 radian storey drift and gap opening between the end plates became more pronounced. One bolt on the top stub side became loose during the second positive excursion of this drift level, indicating that significant elongation occurred in the bolt during the previous negative excursion. Web buckling became apparent during the first cycle at 0.04 radian storey drift and triggered the strength deterioration of the specimen. Figure 4.5 illustrates the progression of link deformation through the test. Close-ups of buckled flanges and webs are shown in Figure 4.6.

Cracking initiation was observed at the top weld access hole on the stub side during the 0.05 radian cycle. During the subsequent cycles at 0.04 radian storey drift, small cracks were also observed in the top and bottom flanges near the weld on the stub side. These cracks propagated very slowly and closed upon load reversal. Figure 4.7 shows the crack locations and sizes.

After the test was terminated and the specimen was returned to its zero position, several bolts remained lose. The flanges and web of the link remained permanently buckled and the link axially shortened by 17mm.

The beam, beam-stub, and column did not experience significant yielding. Minor flaking on the beam web was first detected near the ends of the stiffeners at 0.0075 radian storey drift and propagated into the web area adjacent to the end plate. Flaking was also observed on the bottom flanges of the beam stub as well as at the interior corners of the panel zone during the 0.03 radian cycles. Yielding of these components stopped propagating after 0.03 radian storey drift. Close-ups of these connecting components are shown in Figure 4.8.
Figure 4.3  Summary of significant events - specimen MRF-2A

A: 0.01-1P  Yielding observed on link flanges
B: 0.015-1N  Minor gap opening of stub side connection detected at top link flange location
C: 0.02-1P  Yielding propagated into web
D: 0.03-1P  Link flange buckling observed
E: 0.04-1P  Link web buckling observed
F: 0.05-1N  Crack observed at top weld access hole, on stub side
G: 0.04-3N  Crack observed on top flange, near weld on stub side
H: 0.04-4P  Crack observed on bottom flange, near weld on stub side
I: 0.04-4N  Test terminated
Figure 4.4    MRF-2A, 0.05 rad

(a) 0.015 rad
(b) 0.03 rad
(c) 0.04 rad
(d) 0.05 rad

Figure 4.5    MRF-2A link close-up
Figure 4.6  MRF-2A: (a) web and bottom flange buckling, (b) top flange buckling
Figure 4.7  MRF-2A cracking: (a) top weld access hole, (b) top flange near weld
Figure 4.8  MRF-2A (a) end plate gap opening, top stub side (b) yielding in beam-stub, (c) yielding in beam, (d) yielding in panel zone
Global Response

The load vs. displacement hysteretic behavior is presented in Figure 4.9. The peak load reached is 479kN, during the first positive excursion at 0.04 radian storey drift. Strength of the specimen began to deteriorate after reaching this load and dropped below 80% of the peak load during the first negative excursion at 0.05 radian storey drift. The beginning of this strength deterioration coincided with the beginning of web buckling in the link, which accelerated link flange buckling. Slight pinching due to bolt slippage is observed in the hysteretic curve. The specimen successfully completed two cycles at 0.04 radian storey drift without significant load deterioration, therefore the acceptance criteria for this specimen was met.

Figure 4.10 plots the moment at the column face against the total plastic rotation. The specimen reached 0.031 radian total plastic rotation in the last complete cycle before its strength deteriorated below 80% of the peak load. The maximum total plastic rotation achieved prior to termination of the test, without significant fractures in the link, was 0.041 radian. The maximum $M_{cf}$ experienced by the specimen is 1.13 times greater than the capacity design value for $M_{cf}$ calculated using the probable link capacity ($C_{pl}R_pM_{pl,link}$). The value of $M_{cf}$ did not deteriorate below the value calculated using the design strength of the link ($0.9M_{pl,link}$) before termination of the test.

Figure 4.11 shows contributions to the total storey drift from different components of the specimen. As the total storey drift increased, the percent contribution from the link also increased, while the percent contributions from the other components decreased. From 0.02 radian drift onwards, the magnitude of drift contributions from the beam, the column, and the panel zone remained essentially the same, indicating that the link has fully yielded and forces in the connecting elements have stabilized. After the strength of the link began to deteriorate due to web and flange buckling, drift contributions from the other components also decreased with the load.
CHAPTER 4. EXPERIMENTAL RESULTS

SEISMIC PERFORMANCE OF STEEL MOMENT-RESISTING FRAMES WITH NONLINEAR REPLACEABLE LINKS

Figure 4.9  MRF-2A load vs. displacement

Figure 4.10  MRF-2A moment at column face vs. total plastic rotation
Local Response

Figure 4.12(a) and (b) shows the strain experienced along the centerline of the link flanges under positive and negative bending, respectively. The blue circles represent strain readings from the top flange, whereas the red circles represent strain readings from the bottom flange. Prior to yielding of the link, the strains were approximately constant along the flanges. As the storey drift increased, strains on compressive flanges did not show significant difference along the length. On the other hand, strains on the tensile flanges were considerably lower near the two end plates than at the center of the link. The differences between the tensile and compressive strain profiles are likely caused by the end plate connections. The thick end plates remained rigid under compression, forcing high compressive strain into the adjacent flange areas. When the flange was in tension, however, connection movement resulting from bolt elongation and slight bending in the end plates reduced the strain in the flange areas adjacent to the end plates. The accumulation of plastic tensile strain is thus smaller than the accumulation of plastic compressive strain near the end plates.
Figure 4.12 MRF-2A Strain along centerline of link: (a) positive bending, (b) negative bending
Figure 4.13 shows the strain experienced across the bottom flange. Initially strains are approximately constant across the flange, but the strain profiles became more erratic after the flanges began to buckle.

![Strain Normalized by Yielding Strain](chart)

*Figure 4.13  MRF-2A Strain across bottom flange*
4.3.2 Specimen MRF-2B

Experimental Observations

The performance and failure mode of specimen 2B were very similar to those of specimen 2A. A summary of significant events is presented in Figure 4.14.

Yielding began in the top and bottom flanges near the end plates (predominantly towards the stub side) at 0.0075 radian storey drift, and propagated towards the link center during the 0.01 radian cycles. Slight gap opening was observed between end plates at the locations of the top link flanges on the stub side during the 0.015 radian cycles. At 0.02 radian storey drift, yielding propagated into the web and sounds of bolt slippage was noticeable at the beginning of load reversals. Flange buckling was first observed at 0.03 radian storey drift, and web buckling occurred at 0.04 radian storey drift, initiating strength deterioration. Two bolts near the top flange became loose during the positive excursion of the second cycle at 0.04 radian storey drift, indicating bolt elongation during the previous negative excursion. Figure 4.16 illustrates the progression of link yielding and deformation through the test. Close-ups of buckled flanges and web are shown in Figure 4.17.

During the 0.04 radian cycles, hairline cracks were observed on the top and bottom flanges near the welds on the stub side. These cracks initially formed at the center of the flanges, directly above the link web. In the subsequent cycles cracks were detected at flange edges near the stub side welds. A crack was also observed in the top stub side weld access hole during the 3rd cycle at 0.05 radian storey drift. Figure 4.18 shows the crack locations and sizes.

After the specimen was returned to its zero position, the flanges and web of the link remained permanently buckled and the link axially shortened by 23mm. The beam, beam-stub, and column panel zone had already experienced slight yielding during test 2A, but yielding did not propagate significantly during test 2B. Because the connection bolts were pretensioned to 70% of their minimum specified strength, gap opening between the connection end plates were less severe than in specimen 2A, as shown in Figure 4.19.
Figure 4.14  Summary of significant events - specimen MRF-2B

**A**: 0.0075-1P  Yielding began on link flanges

**B**: 0.015-1N  Minor gap opening of stub side connection detected on top link flange location

**C**: 0.02-1P  Yielding propagated into web

**D**: 0.03-1P  Link flange buckling observed

**E**: 0.04-1P  Link web buckling observed

**F**: 0.04-1N  Crack observed on top flange, near weld on stub side

**G**: 0.04-2P  Crack observed on bottom flange, near weld on stub side

**H**: 0.05-3N  Crack observed on top weld access hole, on stub side

**I**: 0.06-1N  Test terminated after reaching 0.06 radians on negative excursion
Figure 4.15  MRF-2B, 0.05 rad

Figure 4.16  MRF-2B link close-up

(a) 0.02 rad    (b) 0.03 rad

(c) 0.04 rad    (d) 0.05 rad
Figure 4.17  MRF-2B: (a) top flange buckling, (b) web and bottom flange buckling
Figure 4.18  MRF-2B cracking: (a) top weld access hole, (b) top flange near weld
Figure 4.19  Comparison of gap opening at 0.03 rad
CHAPTER 4. EXPERIMENTAL RESULTS

Global Response

The load vs. displacement hysteretic curve of specimen 2B is also very similar to that of specimen 2A, as shown in Figure 4.20. The specimen reached a peak load of 476kN during the first positive excursion at 0.04 radian storey drift. Strength deterioration began during this cycle, coinciding with the observed link web buckling. Near the peak of the first negative excursion at 0.05 radian storey drift, the strength of the specimen dropped below 80% of the peak load achieved. Strength deterioration stabilized during the 0.05 radian cycles. The peak load achieved is similar to that of specimen 2A. This is consistent with the observations of Fleischman et al. (1991) that for large capacity end plate connections, snug-tight bolts and fully-pretensioned bolts achieve the same connection ultimate strength. No visible pinching is observed in the hysteretic curve, indicating that pretensioning the bolts to the specified level improves the connection performance.

The specimen successfully completed both cycle at 0.04 radian storey drift without significant strength deterioration, thus the acceptance criteria are met.

Figure 4.21 shows the moment at column face vs. total plastic rotation curve. The maximum plastic rotation achieved before significant strength deterioration occurred is 0.031 radian. The maximum $\theta_{pl}$ experienced is 1.12 higher than the predicted value corresponding to the probable strength of the link. Prior to termination of the test, the specimen reached 0.049 radian plastic rotation without significant fracture, and its capacity did not drop below its design strength.

Drift contributions from the beam, the column, the link, and the panel zone are shown in Figure 4.22. In comparison to specimen 2A, the link contribution to specimen 2B was slightly lower, especially in the lower drift levels. This discrepancy might be due to the higher pretension in the connection bolts of specimen 2B. As the storey drift increased, the percent contribution from the link also increased. The magnitude of drift contributions from the three other components stabilized after 0.02 radian storey drift, indicating that the forces they experience were no longer increasing. After the specimen strength began to deteriorate, the beam, column and panel zone drift contributions also decreased.
Figure 4.20  MRF-2B load vs. displacement

Figure 4.21  MRF-2B moment at column face vs. total plastic rotation
Figure 4.22  MRF-2B member contributions to total storey drift

Local Response

Figure 4.23 shows the strain experienced along the centerline of the link flanges under positive and negative bending, respectively. The tensile and compressive strain profiles are similar to those observed in specimen 2A. Nearly constant strains are observed along the link in both tension and compression at lower storey drifts. At higher storey drifts, compressive strain values are slightly lower at the center of the link, whereas tensile strain values are higher at the center of the link than near the end plates. Because the connection gap opening is less severe in specimen 2B than specimen 2A, tensile strains near the end plates are slightly larger than those experienced in specimen 2A.

Figure 4.24 shows the strain profiles across the bottom flange. Strains were approximately constant across the flange initially, but became non-uniform as the flanges began to buckle. Strain gauges 23 and 24 were damaged during the installation process, thus strain information is not available at those two locations.
Figure 4.23 MRF-2B Strain along centerline of link:
(a) positive bending, (b) negative bending
Strain Normalized by Yielding Strain

Figure 4.24  MRF-2B Strain across bottom flange
Uniaxial strain gauges were placed along the length of the continuity plates to investigate their role in transferring link flange forces into the beam. The strain values obtained indicate that the plates remained elastic throughout the test. The strain profiles along the bottom continuity plate were nearly linear, decreasing away from the face of the end plate (Figure 4.25). Initially the strain at each location increased quickly with increased storey drift. As the force in the link stabilized, however, the increase in strain became small.
4.4 SPECIMENS MRF-1A AND MRF-1B

The two double channel link specimens were geometrically identical except for the thickness of doubler plates used in the connection regions. Specimen 1B used 1/4” doubler plates that were fillet welded on all four sides. The plate corners were kept free to avoid tri-axial stress concentrations near the channel K-line. Specimen 1A used 3/8” doubler plates. Because brittle fractures originated from the plate corners in specimen 1B, the doubler plates were fillet welded all around in specimen 1A. Fillet welds were first placed around the corners of each plate, starting and stopping 50mm away from the corner. Welds were then placed along the four edges of the plates. The surface treatment of the two specimens were also different: specimen 1A retained its mill scale, while specimen 1B was blast-cleaned to increase its friction coefficient. For specimen 1B, a packing plate located between the two channels was attached to one of the channel webs by two short welds. On the other hand, packing plates for specimen 1B were only held in place by the 5/8” bolts.

Because axial shortening was not significant in the double channel links, side loading on the loading head during tests 1A and 1B was much smaller than during tests 2A and 2B. As a result, the set-up allowed tests 1A and 1B to reach higher storey drifts. Test 1B followed the loading protocol up to 0.06 radian storey drift, at which point brittle fractures occurred in the specimen. To investigate the frame’s post earthquake performance (without replacement of the link after a design level seismic event), specimen 1A was tested twice. After following the loading protocol and completing two cycles at 0.04 radian storey drift (test 1), the specimen was left with -0.075 radian residual drift and cycled through the loading protocol again until its failure at 0.07 radian storey drift (test 2).

For both specimens, not all of the 5/8” bolts were used to restrain the channel webs. Only the 6 bolts close to the neutral axis were used. Because the top and bottom of the link webs were expected to experience flexural tension and compression, the top and bottom bolts were not installed to avoid undesirable stresses during large plastic deformations.

During the first test of specimen 1A, after the peak was reached during the first negative excursion at 0.015 radian storey drift, the specimen was accidentally loaded beyond 0.02 radian
storey drift. Upon discovery of the error, loading was immediately reversed and continued to follow the loading protocol.
4.4.1 Specimen MRF-1B

Experimental Observations

A summary of significant events is presented in Figure 4.26. The deformed specimen at 0.06 radian storey drift is shown in Figure 4.27.

![Graph showing significant events and cycle number](image)

- **A**: 0.0075-1P Yielding began on link flanges and along doubler plate welds in connection region
- **B**: 0.01-1P First sound of bolt slip
- **C**: 0.015-1P Yielding began in link web
- **D**: 0.02-1P Yielding began around bolt holes in connection region
- **E**: 0.05-1P Visible buckling in link flanges
- **F**: 0.06-1P Visible buckling in link webs
- **G**: 0.06-2P Fracture formed in bottom flange of white-washed channel
- **H**: 0.05-3N Fracture formed in top flange of white-washed channel Test terminated

**Figure 4.26** Summary of significant events - specimen-1B
Yielding initiated in the middle of the top flange edges at 0.0075 radian storey drift. Slight yielding was also observed along the inner welds of the doubler plates. Yielding propagated at a slower rate than in the end plate specimens because the eccentrically bolted connections were more flexible than the end plate connections. During the 0.015 radian cycles, yielding was observed in the web around the 5/8” bolt holes. At 0.02 radian storey drift, the area around the corner bolts in the stub side connection began to yield, and yielding propagated through the connection region. Flange buckling became pronounced during the 0.05 radian cycles. Web buckling was observed during the first cycle at 0.06 radian storey drift. Throughout the test, flange yielding was concentrated in the region between centerlines of the connections. Photos of the link at selected storey drifts during the test are shown in Figure 4.28. Figure 4.29 shows the buckled shapes of the flanges.
Figure 4.28  MRF-1B link close-up

(a) 0.02 rad  (b) 0.03 rad
(c) 0.04 rad  (d) 0.05 rad
(e) 0.06 rad  (f) after fracture
Figure 4.29  MRF-1B: (a) top flange buckling, (b) bottom flange buckling
Figure 4.30  MRF-1B: (a) first fracture-bottom flange, (b) second fracture-top flange

(a) 0.06 rad

(b) 0.06 rad
Bolt slipping was first heard during the first positive excursion at 0.01 radian storey drift. Correspondingly, the load dropped approximately 30 kN. Subsequent bolt slippage resulted in smaller decreases in load.

During the positive excursion of the second cycle at 0.06 radian storey drift, a brittle fracture formed in the bottom flange of the whitewashed channel. The test was continued until another brittle fracture formed in the top flange of the same channel, in the subsequent negative excursion. Strength degradation did not occur before the fractures. Figure 4.30 shows the fractures in the top and bottom flanges.

Yielding in the beam and the beam-stub were minimal. Slight flaking was observed on the tip of the beam-stub bottom flange during the 0.01 radian cycles, and in the subsequent cycles yielding was also detected near the doubler plate welds on the beam and beam-stub connections. No sign of yielding was detected in the column panel zone.

Global Response

The load-displacement curve for specimen 1B, shown in Figure 4.31, exhibits noticeable pinching resulting from bolt slippage. The specimen reached a peak load of 386 kN during the first positive excursion at 0.06 radian storey drift. A brittle fracture formed in the bottom flange of the whitewashed channel during the second positive excursion at the same drift level. The load dropped suddenly, but began increasing again for the rest of the excursion, reaching above 80% of the peak load. During the subsequent negative excursion, the top flange of the same channel also fractured, resulting in significant strength loss.

The specimen completed both cycles at 0.04 radian without strength deterioration, thus successfully meeting the acceptance criteria.

Figure 4.32 shows the moment at the column face vs. total plastic rotation curve. The maximum plastic rotation achieved before the strength deteriorated below 80% of peak load is 0.049 radian, which is higher than those achieved by the end plate specimens. The maximum $M_{cr}$ experienced is 1.06 times the predicted value corresponding to the probable strength of the link. After the second fracture formed, the capacity of the link dropped below its design strength.
Drift contributions from the different components are shown in Figure 4.33. Because the eccentric bolt connections were more flexible than the end plate connections, and because the nominal strength of link 1B was lower than that of the end plate links, link contribution in specimen 1B is larger than those in specimens 2A and 2B at lower drift levels. As the storey drift increased and the link yielded, the percent contribution from the link further increased. At higher storey drift levels the link contributions from all three specimens were comparable. Although the magnitude of drift contribution from the beam, the column, and the panel zone did not stop increasing before the link flanges fractured, they increased at a slower rate after 0.03 radian storey drift.

![Figure 4.31 MRF-1B load vs. displacement](image-url)
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Figure 4.32  MRF-1B moment at column face vs. total plastic rotation

Figure 4.33  MRF-1B member contributions to total storey drift
Local Response

Figures 4.34 and 4.35 show the flange strains along the length of the instrumentation link, close to the channel web. The blue circles represent strains on the top flange, whereas the red circle represent strains on the bottom flange. At lower storey drift levels, the flanges exhibits nearly parabolic strain profiles, with higher strain at the center of the link. This strain profile is expected, because the link experienced decreasing moment in the connection regions, away from the center of the link. The flange strains are also slightly higher on the stub side than on the beam side, because the link experienced higher moment on the stub side.

When the link is under positive bending, the flange strains experienced are lower than when the link is under negative bending. This difference likely results from the position of the link when the connection bolts were pre-tensioned. The overhead crane did not exert upward tension on the beam during the bolt pre-tension procedure, therefore gravity caused the link and the beam to rotate in the negative bending direction, allowing the bolts to bear against the bolt holes. During the positive excursions, the bolts must slip and travel to the other side of the bolt hole before significant deformation can occur in the link itself. As the bolt holes become more deformed at each drift level, the bolts must slip further at the subsequent drift level. During the negative excursions, however, the bolts become engaged much earlier because of their initial position. As a result, the links experience higher bending deformations during the negative excursions than during the equivalent positive excursions.

The strains experienced during the 0.00375, 0.005, and 0.0075 radian cycles are similar for both positive and negative bending, as shown in Figures 4.34(a) and 4.35(a). However, strains did not increase during the negative excursion at 0.01 radian storey drift, coinciding with the beginning of observed bolt slippage. This observation further indicates that the initial positions of the bolts with respect to the bolt holes influence the bending deformation of the link.

At 0.04 radian storey drift and above, the tension flange begins to experience higher strain than the compression flange for both positive and negative bending. This coincides with the beginning of the observed flange buckling.
Figure 4.34  MRF-1B Strain along length of instrumentation channel, positive bending: (a) elastic strain, (b) inelastic strain
Figure 4.35  MRF-1B Strain along length of instrumentation channel, negative bending: (a) elastic strain, (b) inelastic strain
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Figure 4.36  MRF-1B Strain along bottom flange of whitewashed channel

Strain gauges were also placed on the bottom flange of the whitewashed channel to investigate whether deformations of the two channels were symmetrical. Figure 4.36 shows the strain profile along the bottom flange, close to the channel web, during both positive and negative bending. When the bottom flange is in compression, it experiences very similar strains as the bottom flange of the instrumentation channel (Figure 4.35(b)), with slightly higher strains on the beam side. When the link is under positive bending and the bottom flanges are in tension (Figure 4.34(b)), however, the strain experienced by the whitewashed channel is lower than that experienced by the instrumentation channel, especially on the beam side. This difference indicates that connection movements in the two channels were not uniform during the upward excursions of loading.

Figure 4.37 shows the strain along the flanges of the instrumentation channel, close to the flange tip. At lower storey drifts, these strain profiles are very similar to those measured close to the channel web, indicating that the stress distributes evenly across the channel flanges when the link is linear elastic. As the storey drift increases, the buckling profiles of both flanges can be observed in their compressive strain profiles.
Figure 4.37 MRF-1B Strain along length of instrumentation channel, close to flange tip:
(a) positive bending (b) negative bending
4.4.2 Specimen MRF-1A

Experimental Observations

A summary of significant events is presented in Figure 4.38. The deformed specimen at 0.07 radian storey drift is shown in Figure 4.39.

During the earlier cycles of the first test, the behavior of specimen 1A was similar to that of specimen 1B. Yielding initiated at the center of the channel flanges above the web, at 0.0075 radian storey drift. At the same time, yielding was also observed along the welds of the doubler plates. Propagation of yielding was slow because of the flexibility of the connections. At 0.015 radian storey drift, yielding began around the 5/8” bolt holes, in a pattern similar to that observed in specimen 1B. Yielding was also observed around the corner bolt holes in the stub side connection. By the end of the first test, the center of the flanges showed significant yielding. A vertical yield line was observed on the channel web, close to the beam side connection. It coincided with the end of the beam, and was likely due to an initial misalignment of the beam and the beam stub. No noticeable bolt slip was heard.

The second test began with -0.0075 radian residual drift. Bolt slip was first heard during the positive excursion of the first cycle at 0.0075 radian, with a corresponding drop in load. At 0.02 radian storey drift, yielding propagated around the edge bolt holes in the connections. Visible flange buckling was observed during the 0.05 radian cycles. Web buckling was delayed by the 5/8” bolts connecting the two webs together, but was visible at 0.07 radian storey drift. Figure 4.40 shows photos of the link at selected storey drifts. Web and flange buckling are shown in Figure 4.41.
Figure 4.38  Summary of significant events - specimen-1A

A: 0.0075-1P  Yielding began in link flanges and along doubler plate welds in connection region

B: 0.015-1P  Yielding began in link web and connection region

C: 0.00375-1P(2) Protocol restarted with -0.0075 radian residual drift

D: 0.0075-1P(2)  First sound of bolt slip

E: 0.05-1N  Visible buckling in link flanges

F: 0.06-1P  First crack observed around doubler plate corner, on stub side

G: 0.07-1N  More cracks formed around doubler plate corners and through hole for 5/8 inch bolts

H: 0.06-3P  Crack propagated through bottom flanges of the link, first on white-washed channel, then the other channel. Test terminated
In comparison to specimen 1B, the improved welding details for the connection doubler plates in specimen 1A delayed failure and avoided brittle fractures. Cracks initiated around the inside corners of the stub side connection doubler plates, at 0.06 radian storey drift. Over the subsequent cycles, these cracks propagated slowly and additional cracks initiated around the 5/8” bolt holes at top and bottom. The strength of the specimen began to degrade during the second cycle at 0.07 radian storey drift, as the cracks grew. During the positive excursion of the fourth cycle at 0.07 radian storey drift, one of the cracks propagated through the bottom flange of the whitewashed channel. When the test was continued, a crack in the other channel quickly grew across its bottom flange, resulting in significant strength degradation. Details of the cracks are shown in Figure 4.42. Photos of the fractured flanges are shown in Figure 4.43. Propagation of the two major cracks on the whitewashed channel are illustrated in Figures 4.44 and 4.45.

Yielding in the beam and beam-stub did not propagate further during test 1A. The column panel zone remained linear elastic at the end of the test. Figure 4.46 shows the stub-to-column connection at the end of the test. No sign of yielding was detected near the welds.

Figure 4.39   MRF-1A, 0.07 rad
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Figure 4.40    MRF-1A link close-up

(a) 0.02 rad
(b) 0.04 rad
(c) 0.04 rad (second test)
(d) 0.06 rad (second test)
(e) 0.07 rad (second test)
(f) after fracture

Figure 4.40    MRF-1A link close-up
Figure 4.41 MRF-1A: (a) web buckling, (b) bottom flange buckling
Figure 4.42   MRF-1A web cracks
Figure 4.43  MRF-1A bottom flange fractures

(A) east channel bottom flange

(b) west channel bottom flange
Figure 4.44  Propagation of crack 1
Figure 4.45  Propagation of crack 2

(a) 0.07 rad (2nd cycle)

(b) test termination
Figure 4.46   MRF-1A stub-to-column connection, 0.07 rad
Global Response

The load displacement curves of both tests carried out on specimen 1A are shown in Figure 4.47. For the rest of the discussions and figures in this section, the residual drift of test 1 is removed from the results of test 2. The load-displacement curve of specimen 1A’s first test, shown in Figure 4.48(a), is very similar to that of specimen 1B. Significant pinching due to bolt slippage is present. The specimen underwent two cycles at 0.04 radian drift without strength deterioration, reaching a peak load of 351 kN. Figure 4.48(b) shows the load displacement curve for the second test. Prior to 0.02 radian storey drift, the maximum load experienced during each cycle was significantly lower than the maximum loads experienced during equivalent cycles in the first test. Bolt holes in the link connection regions elongated during the first test, thus allowing the specimen to reach the same storey drift without carrying similar loads. Because the residual drift was negative, the specimen began experiencing higher loads on the negative excursions of 0.03 radian cycles. A peak load of 400 kN was reached during the first positive excursion at 0.07 radian storey drift. In the subsequent cycles, cracks in the link web propagated, causing slow deterioration in the link strength. The link strength deteriorated below 80% of the peak load during the third positive excursion at 0.07 radian storey drift.

Both tests met the acceptance criteria, indicating that if the link was not replaced after a design level earthquake, it could still achieve satisfactory performance during a subsequent similar seismic event.

The moment at the column face vs. total plastic rotation curves for both the first and second tests are presented in Figure 4.49. The curve for the first test is again similar to that of specimen 1B. The maximum moment experienced at the column face, at 0.04 radian storey drift, does not exceed the value predicted using the probable strength of the link. The maximum plastic rotation achieved during the second test before the specimen strength deteriorated below 80% of the peak load is 0.059 radian. The maximum \( M_{c,r} \) experienced is 1.10 times higher than the moment corresponding to the probable moment capacity of the link. During the last positive excursion of the test, after cracks tore through the bottom flanges of the link, the capacity of the link dropped below its design value.
Drift contributions from the different components are shown in Figure 4.50. Component contributions in the first test of specimen 1A are very similar to those in specimen 1B. For cycles below 0.02 radian storey drift, the link contribution during the first test of specimen 1A is slightly higher than during test 1B. This is most likely because bolt holes in the beam and beam-stub connection regions elongated slightly during test 1B. Because bolt holes in the link connection regions were deformed during the first test, link contribution to storey drift is significantly higher during the lower cycles in the second test. At higher storey drifts, however, the component contributions are again very similar between all three double channel link tests. The percent contribution from the link increased with the storey drift. At 0.07 radian drift, the link contributed to 93% of the total storey drift. The percentage plastic drift contribution from the link would be even higher, as the beam, the column, and the panel zone remained essentially linear elastic through the test.

![Figure 4.47](image-url)

**Figure 4.47** MRF-1A load vs. displacement, residual drift not removed
Figure 4.48  MRF-1A load vs. displacement: (a) test 1 (b) test 2
Figure 4.49  MRF-1A moment at column face vs. total plastic rotation: (a) test 1 (b) test 2
Figure 4.50  MRF-1A member contributions to total storey drift: (a) test 1 (b) test 2
Small potentiometers were placed between the bottom beam-stub/beam flanges and bottom instrumentation channel flanges to measure connection rotation on both sides of the link (Figure 4.2). The percent contribution to link rotation from both the beam connection and the stub connection during the first test are plotted in Figure 4.51. The values presented are from the first positive and negative excursions at each storey drift level. Results are not available for the second test because severe flange buckling occurred near the points of measurement.

Due to the moment gradient along the length of the link, the contribution from the stub connection was greater than the contribution from the beam connection at all drift levels. Initially, as the storey drift increased and bolts began slipping, the percent contributions from the connections also increased. During the positive excursion at 0.02 radian storey drift, the connections together contributed to 55% of the total link rotation. As more bolts became engaged at higher storey drifts, the link bending deformation became more significant and link rotation due to connection rotation became less significant. This trend is observed for both positive and negative bending of the link and for both connections. Lack of symmetry between the connection contributions at positive drifts and negative drifts are likely due to the initial position of bolts at the time of pre-tensioning and the large scatter that is expected in the bolt slipping loads.

![Figure 4.51 MRF-1A connection contributions to link rotation](image-url)
Local Response

Figures 4.52 and 4.53 show the flange strain profiles along the length of the instrumentation channel, close to the channel web. The blue circles represent strains measured from the top flange, and the red circles represent strains measured from the bottom flange. During the linear elastic phase, these strain profiles are very similar to those of specimen 1B. Unlike specimen 1B, however, the bolts in specimen 1A were installed while the overhead crane still exerted an upward force on the beam. The beam and the link experienced smaller initial negative rotation and fewer bolts were initially bearing against their bolt holes than in specimen 1B. As a result, the levels of strain experienced in specimen 1A during the inelastic cycles are more similar between positive and negative bending than in specimen 1B.

Figure 4.54 shows the strain profiles along the top and bottom flanges of the instrumentation link, close to the flange tip. Flange buckling was not observed during the specimen until the second test, and is also not detected in these strain profiles from the first test.

Figure 4.55 shows the strain profile along the bottom flange of the whitewashed channel, close to the channel web. During negative bending, the strain profile is very similar to that of the instrumentation channel bottom flange. During positive bending, however, the beam side of the whitewashed channel experienced lower strains than the beam side of the instrumentation channel.

Strain gauges 9 and 28 were damaged during installation, thus strain information is not available at those two locations. Strain data gathered during the second test are not presented because the link also carries residual deformations from the first test.
Figure 4.52  MRF-1A Strain along length of instrumentation channel, positive bending:
(a) elastic strain  (b) inelastic strain
Figure 4.53  MRF-1A Strain along length of instrumentation channel, negative bending:
(a) elastic strain (b) inelastic strain
Figure 4.54  MRF-1B Strain along length of instrumentation channel, close to flange tip:
(a) positive bending  (b) negative bending
For both specimens 1A and 1B, an uniaxial strain gage was placed on an end stiffener to investigate their contribution to link performance. The measured strains at selected storey drifts are presented in Figure 4.56. Residual strains from the first test of specimen 1A are not removed from the strains measured during the second test. Because the strain measurements are small, errors arising from signal noise can be significant, which may explain the discrepancy between tests 1A and 1B. A general trend is apparent, however. At lower storey drifts the strain was positive, indicating that the stiffeners initially experiences tensile stresses as it constrains the top and bottom flange tips. As storey drift increased, the strain became negative as the flanges began to buckle inwards. This is consistent with the observed deformation in the link, which indicates that the stiffeners were effective in shortening the individual buckling length of the flanges, thereby delaying the onset of flange buckling.
Figure 4.56  MRF-1A end stiffener strain
4.5 ENERGY DISSIPATION

As the specimens deformed inelastically, they dissipated energy equivalent to the area enclosed by the load-displacement hysteretic loops. Figure 4.58 shows each specimen's cumulative energy dissipation during testing. All specimens underwent the same number of cycles at each storey drift level unless otherwise indicated.

![Cumulative energy dissipation graph](image)

**Figure 4.57  Cumulative energy dissipation**

The end plate links, specimens 2A and 2B, have similar levels of energy dissipation up to 0.05 radian storey drift. Specimen 2A completed one cycle at this drift level, whereas specimen 2B completed three cycles, which accounts for the large discrepancy in the total dissipated energy (739 kN-m and 1273 kN-m, respectively). The last two cycles at 0.04 radian storey drift are not included in the accumulation for specimen 2A, and the cumulative energy dissipation for specimen 2B does not include the last cycle which reached 0.06 radian storey drift on its negative excursion.

Because of the slight pinching observed in the hysteretic curve of specimen 2A, the energy dissipated up to and including the cycles at 0.04 radian storey drift is also lower than that dissipated by specimen 2B. The trend of the two curves are similar. Appreciable energy dissipation began
CHAPTER 4. EXPERIMENTAL RESULTS

SEISMIC PERFORMANCE OF STEEL MOMENT-RESISTING FRAMES WITH NONLINEAR REPLACEABLE LINKS

during the 0.015 radian cycles and increased significantly during subsequent cycles, as the plastic hinge forms within the link.

All three tests for the double channel link specimens show similar levels of energy dissipation. Slight deterioration was observed in specimen 1A between the first and second tests: the amount of energy dissipated during the first test was 407 kN-m after completing two cycles at 0.04 radian storey drift, while only 377 kN-m was dissipated after the same number of cycles during the second test. This deterioration is due to the increased bolt slippage resulting from the link bolt hole elongation from the first test.

Comparable amounts of energy were dissipated by specimens 1A and 1B. Specimen 1B dissipated 878 kN-m of energy after completing 2 cycles at 0.06 radian storey drift, while specimen 1A dissipated 859 kN-m after completing the same number of cycles during its second test. The slopes of the two curves are nearly identical beyond 0.04 radian storey drift. Given that slight deterioration occurred after its first test, specimen 1A would likely have dissipated more energy up to 0.06 radian storey drift during the first test if the test was not terminated at 0.04 radian storey drift. The similarity between these two specimens indicates that their difference in connection doubler plate thickness does not have significant influence on the energy dissipation performance of the specimens. The second test of specimen 1A continued for another three cycles at 0.07 radian storey drift, dissipating 1376 kN-m of energy overall. Specimen 1A’s last positive excursion to 0.06 radian storey drift is not included.

Although the energy dissipation values from the double channel link specimens and the end plate link specimens cannot be compared directly because their design capacities are different, several observations can be made from the trends of the curves. The end plate links dissipated little energy prior to the 0.015 cycles, but as yielding propagated through the cross section of the link, the amount of energy dissipated increased rapidly. On the other hand, the doubler channel links began dissolving energy much earlier due to friction in the connection region. As storey drift increased, however, the rate at which energy is dissipated increased very slowly. A large percentage of the energy dissipation potential is lost because of bolts slipping in ovalized bolt holes. At 0.04 radian
storey drift, as more bolts became engaged before peak displacement is reached during a cycle, yielding propagated through the cross section of the link, resulting in more energy dissipation.

To obtain a more direct comparison between the energy dissipation abilities of the two link types, the energy dissipated during each cycle is normalized using Equation (4.10), where $E_{\text{specimen}}$ is the energy dissipated by the test specimen during a specific cycle and $E_{\text{EPP}}$ is the energy dissipated during an equivalent cycle by an elastic-perfectly-plastic specimen. The loading and unloading stiffness of the elastic-perfectly-plastic loops are calculated from the initial stiffness of each test specimen.

$$E_{\text{norm}} = \frac{E_{\text{specimen}}}{E_{\text{EPP}}}$$ (4.10)

Figure 4.58 shows the normalized energy dissipation, $E_{\text{norm}}$, at each storey drift level after 0.02 radian storey drift. The values presented are averages of $E_{\text{norm}}$ from different cycles at the same drift level. The red circles represent values from the end plate link specimens, whereas the blue circles represent those from the double channel link specimens.
Specimens 2A and 2B show increasing values of $E_{\text{norn}}$ as storey drift increases, because the difference between $E_{\text{specimen}}$ and $E_{\text{EPP}}$ due to the yield transition in the test specimen becomes a smaller percentage of $E_{\text{EPP}}$ as higher displacements are reached. At 0.04 radian storey drift, $E_{\text{norn}}$ reached approximate 80%. Specimen 2A has lower $E_{\text{norn}}$ values than specimen 2B due to the pinching in its hysteretic loops.

The double channel link specimens have $E_{\text{norn}}$ values comparable to those of the end plate link specimens in the earlier cycles. As the storey drift increased, pinching in the hysteretic loops caused the values of $E_{\text{norn}}$ to decrease until they stabilized at around 55%. Bolt slippage in the double channel link connections caused approximately 25% decreased in energy dissipation capacity, in comparison to the end plate specimens. For specimen 1A, pinching was more severe in the second test due to bolt ovalization from the first test, thus at 0.03 and 0.04 radian storey drifts lower $E_{\text{norn}}$ values (approximately 50%) are observed for the second test than for the first test. As specimen 1A began reaching storey drifts not reached during the first test, however, its $E_{\text{norn}}$ values increased to approximately 55%, similar to the stabilized values in test 1B and in the first test of specimen 1A. As more inelastic deformations began to take place in the link flanges and webs at 0.06 and 0.07 radian storey drifts, the $E_{\text{norn}}$ values for specimen 1A continued to increase.

Figure 4.59 compares the normalized energy dissipation between the 1st and 2nd cycle at 0.03 radian for all the test specimens. For both end plate specimens, 2A and 2B, slightly more energy is dissipated during the second cycle because the first cycle continues from the previous negative peak at 0.02 radian, resulting in a smaller hysteretic loop for the first cycle. For specimens 1B and the first test of specimen 1A, the second cycle dissipates approximately 5% less energy than the first cycle due to bolt hole ovalization from the first cycle.

Overall, because the end plate link specimens achieved fuller hysteretic loops than the double channel link specimens during cycles at larger storey drifts, they have superior energy dissipation capabilities than the double channel link specimens.
Figure 4.59  Normalized energy dissipation at 0.03 radian
4.6 INSTALLATION AND REPLACEMENT

4.6.1 Specimens 2A and 2B

After the column was installed, link 2A was lifted in place by an overhead crane and bolted to the beam stub on the column. The beam was then lifted in place by the crane and bolted to the beam stub. Bolts in specimen 2A were tightened by a lab technician using a 1-meter long wrench, not to the minimum specified pre-tension.

After test 2A was completed, the specimen was brought to zero storey drift and the beam was shored for the replacement operation. The bolts were loosened with an air wrench and link 2A was removed using the crane. Because the link axially shortened during the experiment, the beam was slightly shifted in the direction away from the column. Link 2B was lifted in place by the crane and bolted to the beam and the beam stub. Most bolts in specimen 2B were tightened using a calibrated air wrench. Because of the short length of the link, however, the air wrench was unable to reach the four inner bolts in both connections of specimen 2B, thus the long wrench was used to tighten these bolts.

In practice, the link can be bolted to the beam stub in the shop. Only the beam needs to be installed in the field, further simplifying the construction process. Shim plates might be necessary between the end plates, thus their effects on connection performance should be investigated. A replacement procedure similar to the one employed during the experiments can be used in the field: the beam would be shored while the link is replaced using lifting equipment. If residual deformation is present in the structure, the link can be custom fabricated by welding the end plates to fit between the beam and the beam stub.

4.6.2 Specimens 1A and 1B

The double channel links were more labour intensive to install than the end plate links. One channel of specimen 1B was first lifted in place by the overhead crane and temporarily attached to the beam stub using two 1” rods through the bolt holes. The beam was then lifted and also temporarily affixed to the link using two 1” rods. Finally, the second channel was placed opposite the first channel, and both channels were bolted to the beam and the beam-stub. A packing plate
was inserted between the two channels before the bolts were tightened. The bolts were then pre-tensioned using an air wrench. Squirter DTIs, direct tension indicators, were used on selected bolts to calibrate the air wrench, thereby ensuring that minimum specified pre-tension was reached.

![Figure 4.60 Bolts with Squirter DTIs](image)

After the completion of test 1B, the specimen was brought to zero storey drift and the beam was shored. The bolts were loosened using the air wrench and both channels were removed. The two channels of specimen 1A were then installed one by one, held temporarily in place by 1” rods. Packing plates were inserted between the two channels. The channels were then bolted to the beam and the beam-stub using the air wrench. Squirter DTIs were again used on selected bolts.

For specimen 1B, bolts holes on the channel doubler plates and the channels were drilled separately before the doubler plates were welded to the channels. Although care was taken to align the bolts holes on the doubler plates and on the channels, misalignment still occurred during the welding process, effectively decreasing the tolerance in the bolt holes. Due to difficulties encountered during the assembling process for specimen 1B, the doubler plates for specimen 1A were welded to the channels first and bolt holes were then match drilled. As a result, bolts in specimen 1A were easier to install.

In practice, construction and replacement procedures similar to the ones employed during the experience can be used. To decrease construction difficulties in the field, doubler plates should be welded in place before bolt holes are drilled, and the two channels should be match drilled back-to-back. If residual deformation is present in the structure, bolt holes can be custom drilled to match the deformed positions of the beam and the beam stub.
4.7 SUMMARY

This chapter presented experimental results for four MRF beam-to-column subassemblages with nonlinear replaceable links: two with double channel links and two with end plate links. All four specimens met the acceptance criteria outlined in Chapter 3.

The behaviour of the end plate link subassemblages closely resembled that of MRFs with reduced beam sections. Full hysteretic curves were observed. Due to local flange and web buckling, the specimens experienced significant strength degradation (below 80% of peak load achieved during the experiment) after completing both cycles at 0.04 radian storey drift. At this drift level, link deformation contributed to approximate 90% of the total storey drift. No performance degradation was observed in the replacement link in comparison to the original link.

The double channel link subassemblages experienced bolt slippage at the link connections. As a result, pinching was observed in the hysteretic curves. After reaching 0.06 radian storey drift, specimen 1B experienced brittle fractures across its flanges, originating from the doubler plate corners. When improved doubler plate welding details were used in specimen 1A, the link experienced strength degradation resulting from slowly propagating cracks on link flanges and webs. At 0.04 radian storey drift and above, link deformation (including connection deformation), contributed to approximately 90% of the total storey drift. Connection rotation was responsible for up to 55% of the link rotation. Inelastic deformation of the channel flanges were affected by the initial positions of connection bolts relative to their bolts holes at the time of pre-tensioning. Negligible performance degradation was observed in the replacement link in comparison to the original link. The replacement link, specimen 1A, was cyclically loaded according to the protocol twice and met the acceptance criteria on both occasions, indicating that if the double channel link was not replaced after a design level earthquake, it could still provide satisfactory performance during a subsequent seismic event.

The end plate link specimens exhibited higher energy dissipating capacity than the double channel link specimens. Because of their connection flexibility, on the other hand, the double channel links were able to reach a higher storey drift before experiencing strength degradation.
CHAPTER 5:  FINITE ELEMENT ANALYSIS

5.1 INTRODUCTION

Nonlinear finite element models were developed for the beam-to-column test subassemblages using the commercial software ABAQUS. The development of the models is presented in this chapter and the results are validated with the measured responses from the laboratory experiments.

Beam-to-column subassemblages, including RBS and other connections, have been modelled by a number of researchers (El-Tawil et al. (1998), Gilton and Uang (2002), Kim et al. (2002), Zhang and Ricles (2006)). In these models, the beam and column are modelled as a continuous entity, usually with shell elements. Modelling of beam-to-column subassemblages with replaceable links present additional challenges because the bolted connections must also be modelled with reasonable accuracy. Recent advances in computer hardware and software capabilities have made more realistic 3D models possible, especially in the connection regions. The models presented here intended to capture the response of the experimental specimens as closely as possible while still remaining computationally feasible. A number of limitations still exist: material imperfections, residual stresses and strains as well as flaws and defects are not incorporated into the model. Welds and weld access holes are not explicitly modelled in the present study but can be easily incorporated into the models for future parametric study.

5.2 END PLATE LINK MODEL

5.2.1 Geometry and Boundary Conditions

The geometry of the beam, column, and links in the finite element model are identical to the test specimens described in Chapters 3 and 4. The boundary conditions specified are very similar to the actual boundary conditions of the test specimens, as shown in Figure 5.1.
To simulate the stiffened pin connections, rigid plates are created at the column ends to eliminate unrealistic stress and strain concentrations at these locations. These plates are coincident with the locations of the pins on the physical column. Roller boundary conditions are imposed (restrained from movement along the axis of the beam and out-of-plane movements) on the back edge of these plates, simulating the pinned connections. The center line of the bottom plate is restrained from vertical and out-of-plane movements, simulating the effects of the floor beam.

**Figure 5.1 End plate model boundary conditions**
connection in the test set-up. A rigid plate is also created at the end of the beam, coinciding with the location of the loading pin in the test set-up. Vertical displacements are imposed on the horizontal centerline of this plate. Lateral support is provided for the beam at locations where they were provided during the test set-up by restraining the out-of-plane movement of the top and bottom flanges.

The model is subjected to cyclic loading following the step-wise increment of the loading protocol used for the test specimen, with one cycle at each increment level. Because the expected local buckling is not symmetric along the axis of the beam, the entire set-up was modelled.

5.3 Finite Element Mesh

The subassemblage was initially modelled entirely using eight-node reduced integration solid elements (C3D8R elements in ABAQUS). To increase computational efficiency, hybrid models with both solid elements and shell elements were later adopted for the majority of the analyses. In these hybrid models, the column and the majority of the beam are modelled with four-node reduced-integration shell elements (S4R elements in ABAQUS). The links and the connecting areas are modelled using eight-node reduced integration solid elements (C3D8R elements in ABAQUS) in order to accurately capture the connection behavior and the local behavior of the links. The default hourglass control feature in ABAQUS was used to prevent uncontrolled distortion of the mesh, often associated with reduced integration elements.

El-Tawil et al. (1998) and Kim et al. (2002) have previously modelled full beam-to-column connections using reduced integration shell elements and successfully captured local and global instabilities. Mesh sensitive studies by El Tawil et al. (1998) concluded that the inelastic global behavior of the model (W360x382 column section and W920x223 beam section) is not sensitive to mesh refinements above 6 elements across the flange width, and noted that unrealistically high elastic stress concentration may develop if the mesh near the beam flange-to-column flange area is too refined. Kim et al. (2002) achieved satisfactory results using the following meshing scheme for W360x262 column section and W760x147 beam section: 8 elements across the beam flange width, 22 elements down the beam web depth, 12 elements across the column flange width, and 10
elements across the column web depth. The same scheme has been used for the shell portion of the models developed in this study. Because the column and the beam remained essentially linear elastic during the laboratory experiment, they were assigned elastic material properties and used three integration points through the shell thickness. Stresses in these elements were verified after each simulation to confirm that they did not exceed the yield stress of steel.

El-Tawil et al. (1998) created solid sub-models of the intersection between the beam bottom flange and the column (W360x382 column section and W920x223 beam section) to further study the behavior of beam-to-column welds. Elastic convergence study of the sub-models found that with reduced-integration brick elements, even the coarsest mesh with 4 elements across the beam flange results in a reasonable stress distribution in comparison to more refined meshes. After studies on the convergence of the inelastic mesh, 13 elements were modelled across the beam flange and 4 elements were used through the beam thickness. Kim et al. (2002) created detailed solid models of a quarter of the beam-to-column subassemblage (W360x262 column section and W760x147 beam section) to study the stress and strain distribution in the connections. Based on El-Tawil et al. (1998)'s study, Kim et al. (2002) used 4 layers of elements through the flange thickness and 10 elements across the flange half-width. In this study, a slightly coarser mesh was adopted with 3 elements through the flange thickness, 2 elements through the web thickness, 14 elements across the flange width and 18 elements were modelled down the web depth (between the fillet regions). To facilitate mesh transitions at the link-to-end-plate interface, tie constraints were generated at these locations.

The above mentioned mesh configuration was computationally efficient while providing good global response. A more refined mesh can be used for more accurate local stress-strain response. Figure 5.2 shows the finite element mesh of the end plate model.
5.3.1 Material Nonlinearity

The tensile stress-strain properties of the link flange and link web were obtained through laboratory testing and reported in Chapter 4. The measured engineering stress $\sigma_{\text{nom}}$ and strain $\varepsilon_{\text{nom}}$ were converted into true stress $\sigma$ and strain $\varepsilon$ using the relationships defined in Equations (5.1) and (5.2). The true plastic strain $\varepsilon^{pl}$ was then computed according to Equation (5.3).
A combined kinematic/isotropic model was used to represent the material properties in the solid elements, which captures the cyclic inelastic behavior of metals. Due to the lack of cyclic material properties, results from the uni-directional coupon tests are specified as half-cycle test data to approximate cyclic response of the material.

5.3.2 Geometric Nonlinearity

In order to capture the correct buckling shape in the link, initial geometric imperfections were introduced in the model before vertical displacements were imposed on the beam tip. An eigenvalue buckling analysis is first performed on the model with a small reference load at the tip of the beam. A linear weighted combination of the first two buckling modes are then superimposed on the model geometry before the cyclic load-deformation analysis is carried out. The first and second buckling mode of the specimen is shown in Figure 5.3.

El-Tawil et al. (1998) studied the sensitivity of the subassemblage to the size of initial geometric imperfections, and concluded that the effect of the initial geometric imperfection is small. For the present study, analysis was conducted with maximum geometric imperfection of 1mm and 3mm, to determine the extent to which the size of geometric imperfection affect the link behavior. Negligible difference was found in the hysteretic response of the subassemblage.

\[
\varepsilon = \ln(1 + \varepsilon_{nob}) \tag{5.1}
\]

\[
\sigma = \sigma_{nob}(1 + \varepsilon_{nob}) \tag{5.2}
\]

\[
\varepsilon^{pl} = \varepsilon \angle \frac{\sigma}{E} \tag{5.3}
\]
5.3.3 Fixed Link Model

To validate the boundary conditions and material properties used for the end plate link model, the model was first developed with rigidly connected end plates in place of the bolted end plate.
connection. The end plates are attached to each other using tie constraints. The result of the fixed link model was used to find an upper-bound for the load-displacement hysteretic curve and to investigate the effect of the bolted connection on the link response. The fixed link model used all solid elements, and the fillets of the link were not expressly modelled.

The load displacement curve of the fixed link model is shown in Figure 5.4, in comparison with the experimental results of specimen 2B. The buckled shape of the links were captured by the model, as well as the general yielding pattern of the subassemblage. Material nonlinearity was also successfully captured. The initial stiffness of the finite element model is slightly higher than that of the test specimen, because the rigidly connected end plates offer higher stiffness than end plates pretensioned together by bolts. Under larger loading cycles, the elastic stiffness of the test specimen degrades due to elongation of the bolts and hence loss of bolt pretension force. The same phenomenon is not captured by the fixed link model. Hence the bolted connection needs to be modelled to provide a more accurate simulation of the test specimen.

![Figure 5.4](image-url)  
**Figure 5.4** Global response of subassemblage with fixed end plates
5.3.4 Bolt Model

Because the bolts for the end-plate connections act mainly in tension, they were modelled using basic axial wire connectors, as shown in Figure 5.5. These connectors are attached to the outside faces of the end plates, at locations corresponding to bolt centers in the test specimens. Load deformation relationships obtained from laboratory testing of an 1” A490 bolt is assigned to each connector using the kinematic material model.

Figure 5.5  Connectors and associated coupling constraints

Prior to cyclic loading of the specimen, connector loads equal to the specified bolt pretension load is applied to the connectors to introduce pretension to the end plate connections. To prevent
excessive distortion at each attachment node, an area equivalent to the contact surface between the
bolt head and the end plate is couple constrained to the node. This constraint uniformly distributes
the bolt load over the said area and mimics the effect of the bolt head and the bolt nut. The stresses
within the couple constrained area were integrated over the area and verified against the applied
load.

Initially the end plates were assigned linear elastic material properties. However, the stresses in
the end plates were found to be higher than the yielding stress of steel after preliminary analyses.
Because the material properties of the plates were unknown, the end plates were assigned the same
material properties as the link flanges. Figure 5.6 illustrates the stress distribution in the link end
plates. Figure 5.7 shows the different hysteretic response when the end plates are assigned elastic
material properties and inelastic material properties. Softer yielding corner is observed in the larger
cycles.

![Stress distribution in link end plates](image)

**Figure 5.6** Stress distribution in link end plates
Figure 5.7  Effect of elastic vs. inelastic end plates

Gradual loss of pretension was observed in the bolted connections during the laboratory experiments. During the course of the test, several bolts became loose. To investigate the effect of pretension loss on the global response of the specimen, analyses were conducted with bolts which gradually lost pretension. Because the actual pretension loss in the individual bolts are unknown and different levels of loss were experienced by bolts at different locations, the loss was assumed to occur linearly with time. All bolts have an initial pretension force of 285kN. Figure 5.8 shows the results from analyses with constant bolt pretension, decreasing pretension to 150 kN and decreasing pretension to 1kN. Softer yielding corners were observed in the model with decreasing pretensioning force, especially in the later cycles. However, the ultimate loads of the models remained similar, which is consistent with observations by Fleischman et al. (1991) that end plate connections with snug-tight bolts and fully-pretensioned bolts achieve the same ultimate strength.
5.3.5 Comparison with Experimental Results

A final analysis was performed following the complete loading protocol used for the laboratory experiments. The bolt pretension loss was assumed to occur linearly with time from 285kN to 1kN. Good correlation were observed between the global hysteretic response of the analytical model and the experimental specimen, as shown in Figure 5.9. The initial stiffness and the yielding curve match well with the experimental results. The Bauschinger effect, characterized by a reduced yield stress upon load reversal after plastic deformation has occurred during the initial loading (ABAQUS 2006), is observed from the analytical results. Local buckling of the beam flange and web are not as severe as observed during the experiment, resulting in less strength degradation. The maximum load achieved by the model is lower than that achieved by the test specimen. These discrepancies are likely due to the approximate cyclic material properties used for the analyses. The deformations and stresses of the model at 0.05 radian storey drift are shown in Figures 5.10 and 5.11. The runtime for this analysis was 5 days on a Windows XP platform using 1.5G of RAM and a 2992Mhz processor.
Figure 5.9  Global response of subassemblage following loading protocol
Figure 5.10  Deformed shape and Von Mises stress distribution at 0.05 radian storey drift
Figure 5.11  Link flange buckling at 0.05 radian storey drift
5.4 **DOUBLE CHANNEL LINK MODEL**

Additional challenges were presented by the double channel link model, in which the moment and shear were transferred through a number of effects at the bolted web connections. To accurately model this type of connection the following effects must be captured:

i) Nonlinear response of the link, including yielding and buckling under cyclic loads.

ii) Friction between the channel webs and the beam webs. The bolts need to be pretensioned to supply the necessary normal force to the contact surfaces, and a appropriate friction coefficient needs to be defined. Before the connection slips, all moment and shear are transferred through static friction at the connection region. After slip occurs, kinematic friction continues to transfer a constant load as the surfaces slide past each other. This results in a pinched hysteretic response.

iii) Bolt bearing and the resulting bolt hole ovalization. After connection slip occurs, the bolts bear on the bolt holes at large storey drifts, causing localized plastic deformations around the bolt hole. During cyclic loading, bolt hole ovalization during prior cycles cause increasingly pinched response because the bolts need to travel further before engaging with the bolt hole.

The following steps were taken to develop a model that could capture all of these effects:

- A fixed connection model was first developed to verify the nonlinear response of the link and to obtain an upper bound for the hysteretic response.
- A single bolt model was created to verify the modelling approach for bolt pretensions, friction, and bearing
- A simplified global model was analyzed with 4 outer bolts in place in each connection
- A more detailed global model was analyzed with 12 outer bolts in place in each connection.

5.4.1 **Fixed Link Model**

As with the end plate link model, a model with fixed link connections was first developed to validate the response of the link. The boundary conditions as well as shell beam and column meshes used for the fixed double channel link model are identical to those used for the end plate link model. Combined isotropic/kinematic material models are used, similar to those assigned to the end plate
link model. The beam stub and the connection region of the beam are modelled with solid elements to enable further development of the model with contact interactions. A finer mesh is provided in the solid component of the double channel link model than that of the end plate link model, so that more accurate results can be obtained from the contact interactions.

The channel webs are connected to the beam webs using tie-constraints. The packing plate between the channel webs is included in the model, as well as the 5/8” bolts used to prevent individual buckling of the channel webs. The packing plate is 1mm thinner than the space between the channel webs, thus when the 5/8” bolts are pretensioned, a small initial imperfection is introduced in the channels. Figure 5.12 shows the global hysteretic response of the fixed model in comparison with the laboratory test results. The ultimate load is in good agreement, however the bolted connection must be modelled to capture the pinched hysteretic response that was observed in the experiments. Figure 5.13 shows the deformation and stresses experienced by the fixed link model. The 5/8” bolts have been removed from the images to better show the stress distribution in the channels.

![Figure 5.12](image.png)  
**Figure 5.12** Global load-deformation response of fixed link model
5.4.2 Bolt-Contact Interactions

Because the bolt contact model involves three complex phenomena, namely bolt pretension, friction between contact faces, and bolt hole ovalization due to bearing, a simple model with three plates and one bolt was first created to verify the modelling approach. The model is loaded under shear as shown in Figure 5.14. The bolt geometry is identical to the bolts that will be used in the full model. Plate thicknesses are the same as the thickness of the channel and beam webs, including doubler plates.

The combined kinematic/isotropic material model for the A490 bolts was approximated from the monotonic A490 bolt material property obtained from Swanson et al. (2002). To accurately model the connection behavior of the double channel links, contact surfaces were established with the surface-to-surface approach. Because contact simulations are sensitive to numerical problems, a relatively fine mesh had to be provided for the bolt and the plates to allow the analysis to run smoothly: 15 elements were modelled around the bolt holes and 26 elements were modelled around the bolt shank.
For each bolt, there are 5 contact pairs as shown in Figure 5.15. The bolt shank is divided into three sections, which interact separately with bolt holes on the two channels webs and the beam web (including doubler plates identical to specimen 1A). These three sections cause the bolt to act in double shear. The inner faces of the head and the nut are in contact with the doubler plates on the channel webs, and transmit the bolt clamping forces to the channels. In addition to these five contact pairs, contact faces are also specified between the channel webs and the beam web. Frictionless contact is specified for interactions between the bolt shanks and bolt holes. A friction coefficient of 0.30 is used for all other contact faces, similar to the value of 0.33 corresponding to class A surfaces (CSA 2001).

Figure 5.14 One-bolt model boundary conditions and mesh

Figure 5.15 Bolt contact pairs
In the first step of the analysis, the bolt is preloaded to the minimum specified pretension force of 285 kN using the bolt load algorithm in ABAQUS. Displacements are then applied to the outer plates. Figure 5.16 shows the hysteretic response of the analysis. The phenomenon of bolt slip and bolt bearing are both captured by the model. Initially the bolt is positioned at the center of the bolt holes, with equal tolerance on all sides. When displacement is applied, slip first occurs between the moving plates and the fixed plate, until the bolt shank bears against the bolt hole on the fixed plate. Slip then occurs between the bolt head/nut and the moving plates, until the bolt shank also comes into contact with the bolt holes on the moving plates. The bolt is then fully engaged and bolt bearing begins to contribute to the force transfer between the plates. If the yield stress of the plate is exceeded in bearing, ovalization of the bolt holes also take place. During the second cycle, the bolt slips for a longer distance before it is fully engaged, indicating that permanent bolt hole elongation occurred during the first cycle. Thus the phenomenon of bolt hole ovalization is also captured by the model.

![Figure 5.16 Cyclic response of one-bolt model](image-url)
5.4.3 Full Model Attempt

Upon successful implementation of the bolt contact interaction in the one-bolt model, an attempt was made to analyze the subassemblage with all 46 bolts in place (as shown in Figure 5.17). Due to the complexity of the resulting model and the large number of contact surfaces involved, as well as the limitation of available computing hardware, the model did not run successfully. Thus simplified models with 8 bolts and 24 bolts were developed.

![Figure 5.17 Double Channel Link Model with all Bolts in place](image)

5.4.4 Simplified 8-Bolt Model

Two approaches were taken to simplify the double channel link model: reducing the number of elements and reducing the number of contact faces. Because the shell-element beam and column are modelled using linear elastic material properties, their response can be easily approximated. Thus these components were removed from the model and the beam stub face initially connected to the column was given fixed conditions. To achieve the correct loading boundary condition, a rigid handle bar was attached to the solid beam section, and given the appropriate EI value to
compensate for the deformation contribution from the beam and the column. The packing plate as well as the 5/8” bolts restraining the channels webs were not included in the analysis. The resulting geometry and boundary conditions are shown in Figure 5.18.

Figure 5.18  Geometry and Boundary Conditions of Simplified Model

The method of instantaneous center of rotation is commonly used for both slip analysis and ultimate strength analysis of eccentric bolted connections (Kulak et al. 1987). The slip analysis is based on the assumption that at the slip load of the connection, the maximum slip resistance of each individual fastener is reached. On the other hand, the ultimate strength analysis assumes that the fastener deformation and the resulting shear load on the fastener varies linearly with its distance from the instantaneous center. The four corner bolts in each connection are farthest from the instantaneous center of rotation, thus carry most of the shear load after connection slip occurs. The 8-bolt simplified model includes only these four corner bolts in each connection, which significantly
reduces the number of contact interactions. As a result this model is much less computationally intensive and was used to verify that friction, connection rotation, and bolt bearing are captured in the model. A coefficient of friction of 0.50 corresponding to class B surface (CSA 2001) was specified for contact surfaces between the channel webs, the beam webs, and the bolt heads/nuts.

The model was subjected to step-wise increasing loading following the loading protocol used for the experimental program, with one cycle for each step increment up to 0.04 radian. The analysis was completed in 5 days on a Windows Vista platform using 5G of RAM and a 2.66 GHz processor.

![Figure 5.19 Geometry and mesh of 8 bolt model](image)

Figure 5.20 shows the load deformation response of this analysis. The effects of bolt slipping and bolt hole ovalization are successfully captured. After every load reversal, slip first occurs in the beam-stub connection, which experiences higher moment. After two of the bolts become engaged, the load once again increases with the displacement, until slip occurs in the beam connection. More bolts successively come into contact with their bolt holes as local deformation occurs in the previously engaged bolts and bolt holes, until all bolts are fully engaged. Due to bolt hole ovalization, the connections must slip further after each successive cycle before bearing contact is achieved.
Because only four bolts are modelled, the load at which connection slip occurs is much lower than that experienced during the laboratory tests. Furthermore, because the majority of the global deformation is contributed by connection rotation (including connection slip, bolt deformation, and bolt hole ovalization), the link never yields and the ultimate load achieved is much lower than the experimental response.

5.4.5 Simplified 24-Bolt Model

Upon successful completion of the 8-bolt model analysis, eight additional bolts along the outer perimeter of each connection were added to the model to achieve a closer representation of the test set-up, as shown in Figure 5.21. As previously mentioned, these bolts are further away from the instantaneous center of rotation, thus experience higher shear forces during bolt bearing than the inner bolts. Since all bolts are assumed to experience the same load at the moment of slip, fewer bolts result in a smaller connection slip load. Thus the coefficient of friction of 0.33, corresponding to the class A surface on specimen 1A (CSA 2001), was adjusted to 0.44 to compensate for the...
omission of the center bolts and to achieve a similar slip load as the full connection. The adjustment was based on the ratio of the slip load for the 12 bolt connections versus that for the 20-bolt connections, calculated using the method of instantaneous center of rotation.

A cyclic analysis was carried out for the model. Because the analysis was anticipated to be very computationally expensive, a simplified loading protocol was specified, with 1 cycle at 0.01 radian, 1 cycle at 0.02 radian, 2 cycles at 0.03 radian, and 2 cycles at 0.04 radian. The analysis was completed in 28 days on a Windows Vista platform using 5G of RAM and a 2.66 GHz processor. The load displacement response is shown in Figure 5.22.
Figure 5.22 Load displacement response of 24-bolt models

The elastic stiffness and the initial slip load agree well between the finite element analysis results and the experimental results. However, fuller hysteretic curves were predicted by the finite element model than that achieved by the test specimen. This discrepancy is likely due to several causes:

- At 0.04 radian, the connection slip load of the test specimen dropped approximately 70kN from the initial slip load. This decrease in connection frictional resistance is likely due to the loss of preload in the bolts. The bolt preload is physically achieved by turning the nut, thereby introducing a small axial elongation in the bolt. Bendigo et al. (1963) and Fisher et al. (1963) observed that when a bolt loaded in shear approaches its ultimate load, the relatively large shearing deformation experienced by the bolt releases this axial elongation. As a result, the preload and hence the frictional resistance is negligible when the ultimate load is reached (Kulak et al. 1987). Although the bolts do not reach their shear rupture strength in the test specimen, the outer bolts experience some inelastic shear deformation at larger storey drifts, which in turn reduces their preload. On the other hand, the bolt load algorithm used in the finite element model maintains constant preload, which over-estimates the frictional resistance and the slip load at the larger cycles.
• The coefficient of friction between the bolt heads/nuts and the channel webs is likely lower than 0.33, the value corresponding to class A surfaces. Using this coefficient of friction for these contact interactions does not affect the initial slip load, which occurs between the channel webs and the beam web (on the beam stub). However it likely over-estimates the overall frictional resistance of the connection.

• Because the frictional resistance is over-estimated, more deformation is driven into the double channels. As a result, more bending deformation in the channels and less bearing deformation in the bolt holes occur in the finite element model than in the test specimen. For this reason, the cyclic increase in slip is not as significant in the finite element model.

• Because more bending deformation is experienced in the channels in the finite element model, the load increases faster with respect to the global displacement after both connections slip, until yielding began to occur in the channel links.

• Monotonic material properties of the doubler plates as well as the beam webs were unknown, and were approximated using the channel web material properties.

• The difference in loading sequence between the finite element model and the experiment may also contribute to the differing responses. Only 6 cycles are completed for the finite element model, instead of the 30 cycles completed for the test specimen up to 0.04 radians.

• Finally, bolted connection behavior typically has a wide range of variability (Kulak et al. 1987), especially when a large number of bolts are involved.
5.5 SUMMARY

Finite element models were developed for the beam-to-column subassemblies to simulate laboratory results.

A hybrid model with both shell and solid elements were developed for the end plate subassemblage. The bolted connections were simulated using pretensioned wire connector elements. Good correlations were observed between analytical and experimental results for the end plate model. The effects of bolt pretension loss and end plate flexibility was studied and was found to decrease the system stiffness at larger storey drifts under cyclic loading.

The model can be further improved by using more accurate cyclic material properties, finer mesh in the critical locations, and inclusion of more details such as the weld access holes. Using these modelling techniques, parametric analyses can be conducted to further investigate the effects of various parameters on the performance of the links and connecting elements. More rigorous sensitivity analyses should be performed to ensure accurate stress and strain predictions at critical locations for parametric analyses.

Due to the complexity of the double channel link model, a hybrid model with fixed link connections was first developed to find an upper bound on the hysteretic behavior and to verify the nonlinear response of the links. A one-bolt model was used to develop the friction and bolt-bearing modelling approaches for the bolted connections. A simplified model with 8 outer bolts was then developed for the double channel links to verify these modelling approaches. Finally, the model was expanded to include the 24 most critical bolts of the connection region. The elastic stiffness and the initial slip load were well captured. The hysteretic loops were fuller than the experimental response, predominantly because gradual loss of bolt preload due to shear deformations was not taken into account.

Further refinement of the model is required before the model can be used to perform parametric analyses, taking into account the decreasing bolt preload and using a more accurate estimate of the coefficient of friction between the bolt heads/nuts and the channel webs. Due to the large number of contact interactions involved, this model is very computationally expensive. In order to perform parametric analyses for the double channel links, a more powerful computing platform is necessary.
CHAPTER 6: DESIGN RECOMMENDATIONS

Based on successful experimental verification of the replaceable link concept, the following preliminary design guidelines are proposed. Because limited test results are available, a number of the design parameters deserve further investigation, as noted within the recommendations.

First, observations pertaining to the overall design of moment frames with replaceable links will be discussed. Then a step-by-step design guideline will be presented for the beam-to-column connection region. Finally, guidelines will be provided for detailed design of the link connections. Detailed design for the experimental specimens can be found in Appendix B, although the design guidelines presented herein were improved from the initial design based on experimental results.

6.1 FRAME DESIGN

Preliminary beam and column sizes in moment frames with replaceable links can be selected according to traditional methods of design to meet strength requirements. In seismic regions, it is often necessary to further increase these beam and column sizes to meet storey drift limits. Because the replaceable link concept uncouples the design of the plastic hinge from the design of the beam, link sizes do not need to change when beam sizes are increased for drift design. On one hand this results in significantly lower capacity design forces for the rest of the structure, including slabs and foundations. On the other hand this causes some elastic stiffness loss in comparison to an equivalent moment frame without replaceable links.

Appendix A compares the design of a 5-storey replaceable link moment frame with an equivalent RBS moment frame. For the RBS moment frame, the beam sizes were increased significantly to meet drift requirements. As a result the plastic hinge capacity at each connection more than doubled. Because of the increased capacity design forces, column sizes had to be further increased to meet the strong-column-weak-beam philosophy. Foundation and slab designs are also affected by this increase proportionally.

For the replaceable link moment frame, the link sizes were increased slightly to help meet drift requirements but the plastic hinge capacity at each connection was still 50% less than that of the
equivalent RBS connection. With these link sizes, slightly heavier beams were required to meet drift requirements than in the case of the RBS moment frame. However, the designer can choose to further increase the link sizes to reduce drift and still benefit from significant reduction in capacity design forces.

RBS connections also result in frame stiffness reduction and increased elastic drifts. Due to the variable cross section of the RBS region, accurate prediction of the stiffness loss for each frame is a lengthy process. Thus for design purposes Moment Connections for Seismic Applications (CISC 2005) assumes 7%-9% elastic drift increase for 40%-50% flange reduction. Replaceable links, on the other hand, can be easily incorporated into frame models to accurately assess the building’s fundamental period, base shear, and drift, because each link has constant cross section along its length.

6.2 General Design for Moment Connection

To allow the frame to return to service with minimal requirement for inspection and repair, the connection should be designed to only allow flexural yielding to occur in the link region. The beam, column, panel zone, and beam-to-column joints should be proportioned so that no significant yielding occurs in these elements. A step-by-step procedure is presented below. Much of the procedure follows Moment Connections for Seismic Applications (CISC 2005) and conforms to CAN/CSA-S16-01 (CSA 2001). Due to limited test data, the recommendations are valid for:

- column sizes up to W360
- beam sizes up to W690
- link sizes up to W460
- link-to-beam moment capacity ratio $\frac{M_{\mu,link}}{M_{\mu,beam}}$ of 0.45.

Step 1 Choose location, length, and size of link

Select the distance from the link connection to the column face, $a$, according to the limits recommended in Equation (6.1). These limits are suggested for RBS connections in CISC (2005). The lower limit is intended to permit stress in the plastic hinge to spread uniformly across the beam at the face of the column while...
the upper limit minimizes the increase of moment between the plastic hinge and the face of the column (Moore et al. 1999).

\[
0.5 b_{beam} \leq a \leq 0.75 b_{beam} \quad (6.1)
\]

Size the link so that its factored resistance is greater than the factored load demand at a distance \(a\) away from the column face.

Select the link length, \(s\), according to Equation (6.2). These limits are based on recommendations for RBS connections (CISC 2005), however the depth of the link is considered instead of the depth of the beam. The lower limit is intended to avoid excessive inelastic strains within the plastic hinge (Moore et al. 1999), while the upper limit is intended to avoid significant frame stiffness loss.

\[
0.65 d_{link} \leq s \leq 0.85 d_{link} \quad (6.2)
\]

The locations of \(a\) and \(s\) are shown in Figure 6.1.

Confirm that the beam and the column are adequate for all load combinations specified by CAN/CSA-S16-01 (CSA 2001) and that the design storey drift meets the applicable limits specified by NBCC (NRC-IRC 2005). The links must be expressly modelled with length \(s\) to obtain accurate storey drifts.

Figure 6.1 Location and length of (a) end plate link, (b) double channel link
Step 2  Calculate the probable moment capacity of the link

The probably moment capacity of the link should be determined according to Equation (6.3). The equation is stipulated for RBS connections by CISC (2005). Instead of the plastic section modulus of the reduced beams section, the plastic modulus of the link section is used.

\[ M_{pr} = C_{pr} R_y F_y Z_{link} \]  

(6.3)

where:

- \( M_{pr} \) = probable peak plastic hinge moment
- \( C_{pr} \) = a factor to account for the effects of strain hardening, local restraint, additional reinforcement, and other connection conditions.
- \( R_y \) = 1.15
- \( F_y \) = expected yield stress of the beam sections as defined in Clause 27.1.7 of S16-01.
- \( Z_{link} \) = the plastic modulus of the link section.

Step 3  Calculate shear at link location

Determine shear at the plastic hinge (link location), \( V_h \), according to Section 3.2 of Moment Connections for Seismic Applications (CISC 2005).

Step 4  Calculate strength demands at critical sections

Determine shear and flexural strength demands at the column face and column center according to Section 3.3 of Moment Connections for Seismic Applications (CISC 2005).

Step 5  Capacity Design Beam

Yielding in the beam should be precluded by satisfying Equation (6.4). Beams should be at least Class 2 sections. For RBS connections, CISC (2005) requires that the plastic moment capacity of the beam based on expected yield stress be greater than the probable moment demand at the column face, to prevent connection weld.
fractures. For the replaceable link concept, however, it is desirable to prevent yielding at the connection, thus a more rigorous design criteria is specified.

\[
\phi F_y Z_b \geq M_{cf}
\]  \hspace{1cm} (6.4)

where \( M_{cf} \) is the moment at column face corresponding to the probable moment capacity of the link, calculated in Step 4.

---

### Step 6 Capacity Design Column

Verify that the columns meet capacity design requirements according to Clause 27.2.3.2 of CAN/CSA-S16-01 (CSA 2001).

### Step 7 Panel zone shear design

Yielding in the panel zone should be precluded by satisfying Equation (6.5). If necessary, doubler plates should be provided in accordance with Clause 27.2.4.3(a) of CAN/CSA-S16-01 (CSA 2001). 

*Equation (6.5) is based on Clause 27.2.4.2(b) of CAN/CSA-S16-01 (CSA 2001). The more conservative equation is used to calculate the panel zone shear resistance, to prevent yielding in the panel zone.*

\[
V_r \geq V_{pz}
\]  \hspace{1cm} (6.5)

where

\[
V_r = 0.55 \phi d_c w' F_{yc}
\]

\[
V_{pz} = \frac{M_c}{d_b Z_b t_b}
\]

\( M_c \) = the moment at column face corresponding to the probable moment capacity of the link, calculated in Step 4.

### Step 8 Continuity plate design

Column continuity plates should be provided according to Section 3.5 of Moment Connections for Seismic Applications (CISC 2005).
6.3 END PLATE LINK DETAILED DESIGN

Step 9  Design bolted connection

Typical details for an eight bolt extended end plate connection are shown in Figure 6.2. A490 high strength bolts should be used. Although the moment and shear demands are different at the two ends of the link, both connections should be designed for the demands at the end closer to the column face.

Bolt tension failure should be precluded by selecting bolt size and placement capable of resisting $M_{pr}$, by satisfying Equation (6.6). The equation assumes that the connection strength based on bolt tension rupture is equal to the static moment of the bolt strength about the centerline of the compression flange (Summer 2003). Bolt prying forces are assumed to be insignificant due to the thick end plates used. Similar equations are used for the four bolt extended unstiffened connection and the eight bolt extended stiffened connection in Moment Connections for Seismic Applications (CISC 2005). The connection is designed to resist $M_{pr}$ instead of $M_{ef}$ because the highest moment it will likely experience is the probable moment capacity of the link.
where $h_0$ and $h_1$ are geometric parameters shown on Figure 6.2.

Bolt shear failure should be precluded by satisfying Equation (6.7). The *bolt shear strength of the connection is assumed to be provided by the bolts at the compression flange* (AISC 2005b). $V_h$ is the highest shear force experienced at the connection, instead of $V_{cf}$ in the case of bolted beam-to-column connections. Similar equations are used for the four bolt extended unstiffened connection and the eight bolt extended stiffened connection in Moment Connections for Seismic Applications (CISC 2005).

$$6A_b(0.5F_u) \geq V_h \tag{6.7}$$

**Step 10 Size end plate**

To preclude end plate flexural yielding, the end plate thickness $t_p$ should satisfy Equation (6.8). The equation is based on research conducted by Sumner (2003), which used yield line analysis to determine the strength of end plates. The controlling yield line pattern is shown in Figure 6.2. Because the this yield line pattern is the same as the pattern for the four bolt extended unstiffened connection, the equation used for the latter in Moment Connections for Seismic Applications (CISC 2005) is adopted.

$$t_p \geq \frac{M_{pr}}{0.8F_{yp} \left\{ (d_{1ink} \angle p_i) \left[ \frac{b_p}{2} \left( \frac{1}{p_f} + \frac{1}{g} \right) + \left( p_f + s \right) \frac{s}{g} \right] + \frac{b_p}{2} \left( \frac{d_{1ink}}{p_f} + \frac{1}{2} \right) \right\} + b_p (d_{1ink} \angle p_i)} \tag{6.8}$$

where

$b_p$, $p_f$, $p_i$, $g$, and $d_{1ink}$ are geometric parameters shown in Figure 6.2

$s = \sqrt{b_p g}$ and

$F_{yp} = 250 \text{MPa}$

End plate shear yielding should be precluded by selecting end plate thickness satisfying Equation (6.9). The equation checks the yielding resistance of the extended portion of the end plate. The equation is also used for the four bolt extended unstiffened connection and the eight bolt extended stiffened connection in Moment Connections for Seismic Applications (CISC 2005).
\[ t_p \geq \frac{M_{pr}}{1.1 F_{y_p} b_p (d_{jink} \geq t_{jink})} \]  

(6.9)

where \( t_{jink} \) is the flange thickness of the link, shown in Figure 6.2

Thicker end plates should be selected for the beam and the beam stub to ensure that they are stronger than the link end plates.

**Step 11 Stiffeners**

Stiffeners with thickness similar to the thickness of the link flanges should be provided as continuity plates into the beam and beam stub, as shown in Figure 6.3. The stiffeners in the beam should extend a distance equal to half of the beam depth.

Figure 6.3 also shows typical welding details of the connection. For weld requirements for panel zone doubler plate and column continuity plates, refer to Sections 7.2 and 7.3 of Moment connections for Seismic Applications.
6.4 DOUBLE CHANNEL LINK DETAILED DESIGN

Step 9  Design bolted connection

The eccentric bolted connection should be designed using the method of instantaneous center of rotation, with the load-deformation relationship developed by Kulak et al. (1987) as shown in Equation (6.10). Although the moment and shear demands are different at the two ends of the link, both connections should be designed for the demand at the end closer to the column face.

\[
R = R_0 (1 - e^{\mu \Delta})^\lambda
\]

(6.10)

where \( \mu = 10.0, \lambda = 0.55, \Delta_{\text{max}} = 0.34'' \)

To reduce the bearing deformation of the bolt holes, the bearing capacity of each bolt should be evaluated according to Equation (6.11). Doubler plates can be provided in the connection region if the channel web thickness is not adequate for bearing. The equation is based on Clause 13.10 (c) of CAN/CSA-S16-01 (CSA 2001), and is derived through the observation that the ratio of bearing stress to the ultimate tensile strength of the plate is in the same ratio as the end distance of the bolt to its diameter. An upper limit of 3 is imposed on the end distance to bolt diameter ratio because of limited test results. Mansour et al. (2006), however, found that for eccentric bolted connections with thin plates, Clause 13.10 (c) of CAN/CSA-S16-01 (CSA 2001) results in excessive deformation in front of the loaded edge of the bolt hole, which adversely impacts the performance of the connection and the double channel links. Thus a coefficient of 2 is recommended instead of 3.

\[
B_r = 2 \phi b t d F_u
\]

(6.11)

Step 10 Web-stitching

Web stitching and packing plates should be provided between the two channels to prevent local web buckling of the individual channels. No bolt holes should be drilled in the top and bottom 1/4 of the channel web to prevent undesirable stress concentrations. Further research should be conducted to study the performance of back-to-back links without web-stitching and packing plates, as well as the optimal stitching configuration.
Step 11 Design end stiffeners

Full depth end stiffeners should be provided with a width approximately equal to $b \angle w$ and a thickness of no less than $0.75w$ or 10mm. The recommendations for the end stiffeners are based on guidelines for links in eccentrically braced frame in Clause 27.7.5.2 of CAN/CSA-S16-01 (CSA 2001). The stiffeners are intended to control flange buckling in the links. The welds for the EBF link end stiffeners need to develop the full stiffener yield capacity because of high forces that must be transferred through the stiffeners. In this case, only the minimum weld size needs to be provided. Further research should be conducted to investigate the effects of removing the end stiffeners.

Figure 6.4 shows typical welding details of the connection. For weld requirements for panel zone doubler plate and column continuity plates, refer to Sections 7.2 and 7.3 of Moment connections for Seismic Applications (CISC 2005).
CHAPTER 7: CONCLUSIONS

7.1 SUMMARY AND CONCLUSIONS

In this study, the concept for moment connections with replaceable links was developed and validated. The moment-resisting lateral frames of a five-storey building located in Victoria, British Columbia was designed using this concept. A ground-floor exterior moment connection from this building was then designed with two different types of link details, and tested in a full scale experimental program. Finite element models were developed for both link types to capture the responses observed during the experiments. Finally, preliminary design guidelines were proposed.

7.1.1 Design of Moment Frames with Replaceable Links

Because the replaceable link concept uncouples the design of the plastic hinge from the design of the beam, link sizes do not need to change when beam sizes are increased for drift design. On one hand this results in significantly lower capacity design forces for the rest of the structure, including slabs and foundations. On the other hand this causes some elastic stiffness loss in comparison to an equivalent moment frame without replaceable links, and may require larger beam and column sizes to meet drift requirements. The designer can optimize between reducing capacity design forces and maintaining frame elastic stiffness by controlling the size of the link. Because each link has constant cross section along its length, links can be easily incorporated into frame models to accurately assess the building’s fundamental period, base shear, and drift.

7.1.2 Experimental Study

Two links were tested for each link type, to verify their strength, ductility, and replaceability. The experimental results indicate that these links can provide strength and ductility equivalent to structures built following current practice. All components were welded in the shop and assembled using bolted connections, resulting in rapid construction. In addition, the links were quickly and easily replaced after being tested according to the loading protocol.
End Plate Links

• Strength: The specimens reached the expected level of strength and maintained stable hysteretic response. Significant strength degradation (below 80% of peak load achieved during the experiment) occurred after completing both cycles at 0.04 radian storey drift, meeting the acceptance criteria outlined in Chapter 3.

• Ductility: At 0.04 radian storey drift, link deformation contributed to approximately 90% of the total storey drift. Strength degradation occurred due to ductile local flange and web buckling. Hairline cracks initiated near the link-to-end-plate welds and near the weld access holes. However, these cracks propagated very slowly and closed upon load reversal.

• Energy dissipation: The end plate links exhibited higher energy dissipating capacity than the double channel links. Full hysteretic curves were observed. Little energy was dissipated at lower storey drifts, but as yielding propagated through the cross section of the link, the amount of energy dissipated increased rapidly.

• Replaceability: No performance degradation was observed in the replacement link in comparison to the original link. Although no difficulties were encountered during the replacement procedure in the laboratory, residual drifts in actual structures may present complications in practice. The replacement link can be custom fabricated for the deformed configuration.

Double Channel Links

• Strength: The specimens reached and maintained the expected level of strength. Bolt slippage due to initial tolerances and subsequent bolt bearing deformation caused pinching in the hysteretic response of the subassemblage. Significant strength degradation (below 80% of peak load achieved during the experiment) occurred after finishing a complete cycle at 0.06 radian storey drift or more, exceeding the acceptance criteria outlined in Chapter 3.

• Ductility: Because of the connection flexibility, the double channel links were able to reach a higher storey drift before experiencing strength degradation than the end plate links. Connection doubler plate welding details were found to be critical to avoid brittle fractures across the links flanges. When improved welding details were used, the link was cyclically
loaded according to the protocol twice and met the acceptance criteria on both occasions, indicating that if the double channel link was not replaced after a design level earthquake, it could still provide satisfactory performance during a subsequent seismic event. The links experienced strength degradation due to local flange and web buckling, as well as ductile tearing of the flanges and the webs beyond 0.06 radian storey drift.

- Energy dissipation: The double channel links began dissipating energy at lower storey drifts than the end plate links due to friction in the connection region. At higher drifts, however, a large percentage of the energy dissipation potential was lost because of bolts slipping in ovalized bolt holes. At 0.04 radian storey drift, as yielding propagated through the cross section of the ink, more energy was dissipated.

- Replaceability: No significant performance degradation was observed in the replacement link. The replacement link provided satisfactory performance when cyclically loaded according to the protocol twice. Although no difficulties were encountered during the replacement procedure in the laboratory, residual drift in actual structures may present complications in practice. The replacement link can be custom fabricated for the deformed configuration.

7.1.3 Finite Element Modelling

Nonlinear finite element models were developed for both link types using ABAQUS, with different modelling approaches for the connection regions. Good correlations were achieved between the analytical and experimental results.

End Plate Links

The model for the end plate link subassemblage intended to capture the nonlinear cyclic response of the link as well as the connection region behavior. The entire test subassemblage was modelled, with boundary conditions similar to the physical boundary conditions of the test set-up. Reduced-integration shell elements with elastic material properties were used to represent the beam and the column, while reduced-integration solid elements with nonlinear material properties were used to model the link and the end plates. Connection bolts were modelled using wire connector
elements. They were assigned the measured load-deformation properties of an 1” A490 bolt and pretensioned to the specified pretension load. Investigations found that the bolted connections, including inelastic response in the end plates and reduced pretension force in the bolts, resulted in softer yielding corners at higher cycles, but did not affect the ultimate load reached by the specimen. The resulting global load-displacement response of the model as well as the deformed shape of the model show good agreement with the experimental results.

**Double Channel Links**

The model for the double channel link subassemblage intended to capture the nonlinear response of the channels, friction contact between the connection webs, and bolt bearing deformations. The entire subassemblage was first modelled with fixed link connections to obtain an upper bound estimate of the global response. To increase computational efficiency and to eliminate numerical difficulties, the column and the beam were not included in the model with bolted connections. Solid elements with nonlinear material properties were used to model the links and the connecting regions. A simplified model with 4 outer bolts in each connection successfully captured connection slip and bolt hole ovalization. A more complex model with 12 outer bolts in each connection was modelled to approximate the experimental response, and the coefficient of friction was adjusted to compensate for the decreased slip load due to the missing bolts. The elastic stiffness and the initial slip load were well captured. The hysteretic loops were fuller than the experimental response, predominantly because gradual loss of bolt preload due to shear deformations was not taken into account.

**7.2 Recommendations for Future Research**

Although results from the current study indicate that the replaceable link concept provides good seismic response, additional research is needed to further understand the behavior of the links during an earthquake.
The preliminary models developed in this study may be further refined through material definitions and mesh sizes, to accurately capture local stress-strain behavior and to assess fracture potentials. For the double channel link model, further refinement of the model is required before the model can be used to perform parametric analyses, taking into account the decreasing bolt preload and using a more accurate estimate of the coefficient of friction between the bolt heads/nuts and the channel webs. A more powerful computing platform and more efficient algorithms are necessary to reduce the runtime, in order to make the full model computationally feasible. The explicit approach may be considered as an alternative to the implicit approach used in this study. Van de Vegte and Makino (2004) modelled bolted beam-to-column connections using the explicit approach and achieved reasonable results. For larger models, the explicit approach results in shorter computational time and smoother contact simulations than the implicit approach. However, more rigorous evaluation of the results are required to ensure that the solution is not significantly affected by dynamic effects.

Using the refined finite element models, parametric analyses should be performed to investigate the effects of:

- varying the link length
- using links with greater ratio of link-to-connecting-beam moment capacity
- two-sided connections
- composite slab on the moment frame
- removing beam continuity plates from the end plate connections
- removing end stiffeners from the double channel links
- removing the top and bottom bolt holes connecting the center of the double channel link webs
- varying the eccentric bolted connection bolt pattern for the double channel links
- using replacement links with geometry that reflect the permanent residual drift of the structure

Further experimental investigations may be necessary based on findings from the finite element parametric study. The concept can also be explored as a method of upgrading or repairing existing moment frames.
The replaceable link concept uncouples strength design and drift design of the moment frame, allowing the designer to control the capacity design forces through the size of the links. Smaller link sizes result in smaller capacity design forces but lead to reduced frame elastic stiffness and larger storey drifts, thus requiring larger beams and columns. Increasing the link size helps to control storey drifts but leads to larger capacity design forces. A comprehensive study on the economic impact of varying the link size would be helpful in optimizing the design of moment frames with replaceable links.

Finally, nonlinear time history analyses should be performed for buildings incorporating the nonlinear replaceable concept, to investigate the effect of the links on the overall response of the moment frames.
REFERENCES


REFERENCES

Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.


APPENDIX A: DESIGN OF A MOMENT-RESISTING FRAME WITH REPLACEABLE LINKS

The following calculations outline the seismic design of a moment resisting frame (MRF) with nonlinear replaceable links, in accordance with provisions in the National Building Code of Canada (NRC-IRC 2005) and CAN/CSA-S16-01 (CSA 2001). This design is compared to the design of an equivalent MRF with reduced beam sections. The prototype structure shown in Figure A.1 is a five-storey building located in Victoria, British Columbia, with five 9.0m bays in the direction of the moment frames. The second storey is 4.2m above ground level while all other storeys have interstorey height of 3.7m. The building’s mechanical equipments are housed on the roof, which is a steel deck supported on open web steel joists. The interior floors are concrete on composite deck supported on wide flange beams. The MRFs are symmetrically placed on either side of the building on the perimeter to avoid torsional eccentricity. For this design, all steel is assumed to be CAN/CSA G40.20/G40.21 350W. Table A-1 summarizes the assumed loading for the structure.

![Figure A.1 MRF elevation and building floor plan](image-url)
A.1 DESIGN BASE SHEAR

The following calculations will provide the design base shear for the structure based on the Equivalent Static Force Procedure. This base shear will be used to obtain preliminary member sizes. A dynamic analysis will be then conducted to confirm the design. For type D MRF, the ductility and overstrength factors are $R_d = 5.0$ and $R_o = 1.5$.

A.1.1 Design Spectrum

For buildings located in Victoria, British Columbia, the NBCC provides the following 5% damped spectral response acceleration:

<table>
<thead>
<tr>
<th>Sa(0.2)</th>
<th>Sa(0.5)</th>
<th>Sa(1.0)</th>
<th>Sa(2.0)</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2g</td>
<td>0.83g</td>
<td>0.38g</td>
<td>0.19g</td>
<td>0.62g</td>
</tr>
</tbody>
</table>

Assuming Class C soil, with $F_d = F_v = 1.0$, the design acceleration spectrum for the site condition shown in Figure A.2.
A first estimate of the fundamental lateral period of the structure is calculated according to NBCC 4.1.8.11.3 a):

\[ T_a = 0.85 (h_n)^{0.75} \]  \hspace{1cm} (A.1)

The actual fundamental period is likely much greater. NBCC 4.1.8.11.3 c) states that the period may be taken as up to 1.5 times the solution to Eq. (A.1). To allow convergence to a lower lateral force demand, the fundamental period is increased by the maximum amount allowed. The actual fundamental period will be verified after the analysis is complete. Hence:

\[ T_a = 1.5 [0.85(19)^{0.75}] = 1.16 \text{s} \]

**A.1.2 Building Weight**

The building weight at each level (including 25% of snow load) is calculated and summarized in Table A-2.
**A.1.3 Base Shear Calculation**

The total base shear for the equivalent static force procedure according to NBCC 4.1.8.11.2 is:

\[ V = \frac{S_T(T_s)(M_v)(I_E)(W)}{R_d R_o} \]  \( (A.2) \)

Based on the design acceleration spectrum shown in Figure A.2, \( S_T(T_s) \) is 0.35g. From NBCC Table 4.1.8.11, the higher mode effect \( M_v \) is equal to unity. The importance factor is \( I_E = 1.0 \) because the building is of normal importance. The actual base shear experienced by the structure is likely lower than the one given by Eq. (A.2). Since a dynamic analysis will be performed for the structure, the base shear may be taken as low as 0.8 time the value obtained from Eq. (A.2), according to NBCC 4.1.8.12.5. It must be verified that the base shear obtained from the dynamic analysis is scaled to be at least equal or greater than 0.8 \( V \). To allow convergence to a lower lateral force demand, the maximum allowed reduction will be taken. Thus the design level base shear is:

\[ 0.8 V = 0.8 \left( \frac{0.35(1.0)(1.0)(57246)}{5.0(1.5)} \right) = 2137 \text{kN} \]

**A.2 DISTRIBUTION OF LATERAL FORCES**

**A.2.1 Vertical distribution of base shear**

The design base shear is distributed vertically to each storey in accordance with NBCC 4.1.8.11.6:

\[ F'_x = (V \angle F_s) \frac{W_i h_i}{\sum_{j=1}^{5} W_j h_j} \]  \( (A.3) \)
where \( F_t = 0.07 T_a V = 174 kN < 0.25 V \)

### TABLE A-3  Vertical Distribution of Base Shear

<table>
<thead>
<tr>
<th>Storey</th>
<th>( W_s )</th>
<th>( h_s )</th>
<th>( W_s h_s )</th>
<th>( F_s' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (5)</td>
<td>6368</td>
<td>19.00</td>
<td>120999</td>
<td>559</td>
</tr>
<tr>
<td>4</td>
<td>12706</td>
<td>15.30</td>
<td>194405</td>
<td>619</td>
</tr>
<tr>
<td>3</td>
<td>12706</td>
<td>11.60</td>
<td>147392</td>
<td>469</td>
</tr>
<tr>
<td>2</td>
<td>12706</td>
<td>7.90</td>
<td>100379</td>
<td>319</td>
</tr>
<tr>
<td>1</td>
<td>12760</td>
<td>4.20</td>
<td>53593</td>
<td>171</td>
</tr>
<tr>
<td>Total</td>
<td>57247</td>
<td></td>
<td>616767</td>
<td>2137</td>
</tr>
</tbody>
</table>

### A.2.2 Horizontal distribution of base shear

Since the center of mass coincides with the center of rigidity, and the MRFs are located symmetrically on the outermost bays of the building, the building is unlikely to be torsionally sensitive. Additional base shear due to accidental torsion is accounted for by increasing the storey shear by 10%.

### A.2.3 Notional Loads

The governing load combination is assumed to be 1.0D+1.0E+0.5L+0.25S. In order to account for initial imperfections and partial yielding of frames at factored load level, notional loads equal to 0.005 times the factored gravity loads contributed by each storey must be considered according to S16-01 clause 8.7.2. The notional loads are tabulated in Appendix A-4. Live load has been reduced according to its tributary area in one floor only.

### TABLE A-4  Notional Loads

<table>
<thead>
<tr>
<th>Storey</th>
<th>Specified Dead (kN)</th>
<th>Specified Live (kN)</th>
<th>Specified Snow (kN)</th>
<th>Factored Gravity (kN)</th>
<th>Notional (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (5)</td>
<td>6368</td>
<td>0.00</td>
<td>3062</td>
<td>7134</td>
<td>35.7</td>
</tr>
<tr>
<td>4</td>
<td>12706</td>
<td>2607</td>
<td>0</td>
<td>14010</td>
<td>70.0</td>
</tr>
<tr>
<td>3</td>
<td>12706</td>
<td>2607</td>
<td>0</td>
<td>14010</td>
<td>70.0</td>
</tr>
<tr>
<td>2</td>
<td>12706</td>
<td>2607</td>
<td>0</td>
<td>14010</td>
<td>70.0</td>
</tr>
<tr>
<td>1</td>
<td>12760</td>
<td>2607</td>
<td>0</td>
<td>14064</td>
<td>70.3</td>
</tr>
<tr>
<td>Total</td>
<td>57247</td>
<td>10428</td>
<td>3062</td>
<td>63226</td>
<td>316</td>
</tr>
</tbody>
</table>
A.2.4 P-Delta effect

$P \Delta \Delta$ effects are accounted for by the $U_2$ factor, given in Eq. (A.4). As stipulated in S16-01 clause 27.1.7, structural stiffness shall be provided such that $U_2$ does not exceed 1.4.

$$U_2 = 1 + \left( \frac{\sum C_f R_f \Delta_f}{\sum V_f h} \right)$$ \hspace{1cm} (A.4)

The maximum permissible lateral drift required by NBCC 4.1.8.13.3 for buildings of normal importance is 2.5%. Assuming 2% first order storey drift for each floor:

$$\frac{R_d R_f \Delta_f}{I_E} = 0.020 h$$

Solving for $R_d \Delta_f$ and substituting into Eq. (A.4), $U_2$ can be expressed as:

$$U_2 = 1 + \left( 0.0133 \frac{\sum C_f}{\sum V_f} \right)$$

$C_f$ is the gravity load and $V_f$ is the lateral load including accidental torsion and notional load. Live load has been reduced according to its tributary area in one floor only. This conservative simplification results in slightly larger estimates for $U_2$ in the lower levels. The estimated values of $U_2$ tabulated in Table A-5 need to be verified for the final design.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Factored Gravity (kN)</th>
<th>$\Sigma C_f$ (kN)</th>
<th>$1.1F_x' + \Sigma V_f$ (kN)</th>
<th>$U_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (5)</td>
<td>7134</td>
<td>7134</td>
<td>451</td>
<td>1.15</td>
</tr>
<tr>
<td>4</td>
<td>14010</td>
<td>21143</td>
<td>751</td>
<td>1401</td>
</tr>
<tr>
<td>3</td>
<td>14010</td>
<td>35153</td>
<td>586</td>
<td>1987</td>
</tr>
<tr>
<td>2</td>
<td>14010</td>
<td>49163</td>
<td>421</td>
<td>2409</td>
</tr>
<tr>
<td>1</td>
<td>14064</td>
<td>63226</td>
<td>258</td>
<td>2667</td>
</tr>
</tbody>
</table>

A.2.5 Design Lateral Force

Taking these second order effects and additional base shear due to accidental torsion into account, the final design lateral force at each floor is calculated according to Eq. (A.5) and tabulated in Table A-6.
\[ F_x|_{\text{final}} = (1.1 F'_x + \text{Notional}) U_2 \] (A.5)

The design lateral forces are equally distributed between the two MRFs.

### TABLE A-6 Equivalent Storey Lateral Forces

| Storey | 1.1F'_x + Notional (kN) | U_2 | F_x|final (kN) | F_x|final/2 (kN) |
|--------|-------------------------|-----|-----------|------------|
| Roof (5) | 651                       | 1.15 | 746       | 373       |
| 4      | 751                       | 1.20 | 902       | 451       |
| 3      | 586                       | 1.24 | 724       | 362       |
| 2      | 421                       | 1.27 | 536       | 268       |
| 1      | 258                       | 1.32 | 339       | 170       |

A.3 MOMENT FRAME MEMBER DESIGN

In addition to the seismic lateral forces determined in Table A-6, each moment frame also carries gravity loads on a tributary width of 4.5m. The curtain walls are assumed to be carried by all peripheral columns equally. The combined factored design loads on each frame is shown in Figure A.3. The members are initially analyzed using the portal frame method for the lateral loads and the contraflexure method for the gravity loads. Design iterations are then performed in SAP 2000. For comparison purposes, design was performed for both an MRF with prequalified RBS connections and an MRF with nonlinear replaceable links.

![Figure A.3 Lateral and Gravity Loads on MRF](image)

Note: all loads are in kN
A.3.1 MRF Design with RBS

For this design, 50% flange reduction is assumed for the reduced beam sections. The distance from the column face to the RBS, $a$, is assumed to be 50% of the beam width. The length of the RBS, $s$, is assumed to be 65% of the beam depth. These assumptions are based on recommendations given in Section 6.1(a) of Moment Connections for Seismic Applications (CISC 2005).

The frame is first designed to meet strength requirements. The reduced beam sections are designed to resist the factored demands at the RBS location. The beams and columns are capacity designed to resist the forces associated with yielding of the RBS. However, these member sizes result in interstorey drifts exceeding 2.5% of the storey height, the limit stipulated by NBCC 2005. Thus the beam and column sizes are subsequently increased to meet storey drift limits, as is often necessary for ductile MRFs in seismic regions. The relative stiffness of the storeys are compared to ensure that type 1 irregularity is avoided according to Table 4.1.8.6 of NBCC 2005. The resulting member selections for both the strength design and the drift design are tabulated in Table A-7.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Strength Design</th>
<th>Drift Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam</td>
<td>Column</td>
</tr>
<tr>
<td>5</td>
<td>W310x45</td>
<td>W360x162</td>
</tr>
<tr>
<td>4</td>
<td>W530x66</td>
<td>W360x162</td>
</tr>
<tr>
<td>3</td>
<td>W610x82</td>
<td>W360x162</td>
</tr>
<tr>
<td>2</td>
<td>W610x101</td>
<td>W360x216</td>
</tr>
<tr>
<td>1</td>
<td>W610x101</td>
<td>W360x216</td>
</tr>
</tbody>
</table>

The interstorey drift from the strength design and the drift design are listed in Table A-8 and compared to the interstorey drift limit stated in the NBCC 2005. To reflect realistic values of anticipated deflections based on the equal displacement principal, the lateral deflections obtained from the linear elastic analysis is multiplied by $\frac{R_d R_o}{T_e}$ per Clause 4.1.8.13.2 of NBCC 2005. In addition, the interstorey drifts have been increased by 9% to account for elastic stiffness loss resulting from flange reductions (CISC 2005).
Because the cross section of the RBS is dependent on the choice of the beam, when a beam size is increased for drift design the strength of its RBS also increases. Correspondingly, column sizes need to be increased to ensure that the strong-column-weak-beam requirement is satisfied. In addition, other component of the structure such as slabs and foundations must also be capacity designed. This results in a lengthy design process and often results in overdesigned structures. The probable moment capacities of the RBSs, \( M_{pr} \), from the strength design and the drift design are compared in Table A-9.

### Table A-9  Probable Moment Capacity of RBS

<table>
<thead>
<tr>
<th>Storey</th>
<th>Strength Design ( M_{pr} ) (kN-m)</th>
<th>Drift Design ( M_{pr} ) (kN-m)</th>
<th>%Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>189</td>
<td>842</td>
<td>345</td>
</tr>
<tr>
<td>4</td>
<td>477</td>
<td>1036</td>
<td>117</td>
</tr>
<tr>
<td>3</td>
<td>683</td>
<td>1423</td>
<td>108</td>
</tr>
<tr>
<td>2</td>
<td>842</td>
<td>2063</td>
<td>145</td>
</tr>
<tr>
<td>1</td>
<td>842</td>
<td>2063</td>
<td>145</td>
</tr>
</tbody>
</table>

A.3.2 MRF Design with Replaceable Links

The replaceable link concept uncouples the design of the plastic hinge from the design of the beam, hence link size selection is independent from beam selection. The lengths and locations of the links are chosen based on recommendations for RBS in Moment Connections for Seismic Applications (CISC 2005). The length of the link, \( s \), is based on the depth of the link, and is assumed to be 300mm for the initial design. The distance from the column face to the link, \( a \), is based on the width of the beam, and is assumed to be 125mm. These assumptions are reasonable for the lower floors and conservative for the upper floors.

Similar to the RBS design previously presented in Section A.3.1, the frame is first designed to meet strength requirements. The links are sized to resist the factored loads at the link location, as
shown in Table A-10. The beams and columns are then capacity designed to resist the forces
associated with yielding of the links. However, these member sizes result in interstorey drifts
exceeding the drift limit stipulated by NBCC 2005, and the beam and column sizes are increased for
drift design. Because of increased beam width for drift design, the distance from the column face to
the link, \( a \), is assumed to be 200mm. Again this assumption is reasonable for the lower floors and
conservative for the upper floors.

Because link sizes do not depend on their connecting beam sizes, other members in the frame do
not need to be capacity designed again if link sizes remain the same. In this particular example, the
link sizes are increased slightly to help meet drift requirements. The resulting member selections for
both the strength design and the drift design are tabulated in Table A-7. The relative stiffness of the
floors have been compared to ensure that type 1 irregularity is avoided according to NBCC 2005.

### Table A-10 Demand and Capacity of Replaceable Links

<table>
<thead>
<tr>
<th>Storey</th>
<th>( T_f ) (kN)</th>
<th>( M_f ) (kN-m)</th>
<th>Link Section</th>
<th>( T_r ) (kN)</th>
<th>( M_r ) (kN-m)</th>
<th>( T_f/T_r+M_f/M_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>75</td>
<td>102</td>
<td>W310x28</td>
<td>1121</td>
<td>126</td>
<td>0.87</td>
</tr>
<tr>
<td>4</td>
<td>90</td>
<td>344</td>
<td>W410x60</td>
<td>2354</td>
<td>369</td>
<td>0.97</td>
</tr>
<tr>
<td>3</td>
<td>72</td>
<td>470</td>
<td>W460x74</td>
<td>2934</td>
<td>512</td>
<td>0.94</td>
</tr>
<tr>
<td>2</td>
<td>54</td>
<td>583</td>
<td>W460x89</td>
<td>3540</td>
<td>624</td>
<td>0.95</td>
</tr>
<tr>
<td>1</td>
<td>34</td>
<td>604</td>
<td>W460x89</td>
<td>3540</td>
<td>624</td>
<td>0.98</td>
</tr>
</tbody>
</table>

### Table A-11 Design for MRF with Replaceable Links

<table>
<thead>
<tr>
<th>Storey</th>
<th>Beam</th>
<th>Column</th>
<th>Link</th>
<th>Link</th>
<th>Beam</th>
<th>Column</th>
<th>Link</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>W310x45</td>
<td>W360x162</td>
<td>W310x28</td>
<td>W610x113</td>
<td>W360x347</td>
<td>W410x60</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>W530x82</td>
<td>W360x162</td>
<td>W410x60</td>
<td>W610x125</td>
<td>W360x347</td>
<td>W410x74</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>W610x91</td>
<td>W360x162</td>
<td>W460x74</td>
<td>W690x192</td>
<td>W360x347</td>
<td>W460x97</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>W610x113</td>
<td>W360x216</td>
<td>W460x89</td>
<td>W690x217</td>
<td>W360x421</td>
<td>W460x128</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>W610x113</td>
<td>W360x216</td>
<td>W460x89</td>
<td>W690x217</td>
<td>W360x421</td>
<td>W460x128</td>
<td></td>
</tr>
</tbody>
</table>

Because the links can be expressly modelled, interstorey drifts are accurately assessed using the
SAP 2000 model and tabulated in Table A-12. To reflect realistic values of anticipated deflections
based on the equal displacement principal, these lateral deflections obtained from the linear elastic
analysis are multiplied by \( \frac{R_d R_o}{I_g} \) per Clause 4.1.8.13.2 of NBCC 2005. For both the strength design
and the drift design, two interstorey drift values are presented: one for the frame without replaceable
links, and one for the frame with replaceable links. The percent drift increases due to introduction
of the replaceable links are presented. For strength design, these percent drift increases are not
much higher than the 9\% estimated for frames with RBS connections. The higher increase at the
5th storey is likely due to the conservative assumption of the link length, $s$. For the drift design, on the other hand, because link sizes do not increase with beam sizes, elastic stiffness loss in the frame can be significant. As a result, slightly heavier beams were required to meet drift requirements than in the case of the MRF with RBS connections.

**TABLE A-12** Interstorey Drift of MRF with Replaceable Links

<table>
<thead>
<tr>
<th>Storey</th>
<th>w/o link (mm)</th>
<th>w/ link (mm)</th>
<th>% increase</th>
<th>w/o link (mm)</th>
<th>w/ link (mm)</th>
<th>% increase</th>
<th>0.25h, (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>129.8</td>
<td>149.3</td>
<td>15.0</td>
<td>40.5</td>
<td>54.75</td>
<td>35.2</td>
<td>92.5</td>
</tr>
<tr>
<td>4</td>
<td>157.5</td>
<td>176.3</td>
<td>11.9</td>
<td>60.75</td>
<td>80.25</td>
<td>32.1</td>
<td>92.5</td>
</tr>
<tr>
<td>3</td>
<td>166.5</td>
<td>186.8</td>
<td>12.2</td>
<td>73.5</td>
<td>91.5</td>
<td>24.5</td>
<td>92.5</td>
</tr>
<tr>
<td>2</td>
<td>183.0</td>
<td>204.0</td>
<td>11.5</td>
<td>75</td>
<td>92.25</td>
<td>24.0</td>
<td>92.5</td>
</tr>
<tr>
<td>1</td>
<td>173.3</td>
<td>183.8</td>
<td>6.1</td>
<td>73.5</td>
<td>83.25</td>
<td>13.3</td>
<td>105.0</td>
</tr>
</tbody>
</table>

Although the link sizes were increased to help meet drift requirements and to prevent vertical stiffness irregularity, the resulting probable moment capacities are still much lower than those of RBS connections, as shown in Table A-13. As a result, the capacity demand requirements for all other members in the structure, including slabs and foundations, are significantly lower. Despite the fact that the beam and column sizes are slightly heavier than their RBS counterparts, the replaceable link concept still provides significant economic incentives.

The designer can also choose to optimize and further increase the link size to help control drift, while still being able to enjoy significant reductions in capacity design. Furthermore, since the link is separate from the beam, links with lower yield strength maybe used to achieve higher elastic stiffness while still limiting the capacity design forces.

**TABLE A-13** Probable Moment Capacity of Link vs. RBS

<table>
<thead>
<tr>
<th>Storey</th>
<th>Strength Design $M_p$ (kN-m)</th>
<th>Drift Design $M_p$ (kN-m)</th>
<th>RBS Drift Design $M_p$ (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>180</td>
<td>527</td>
<td>842</td>
</tr>
<tr>
<td>4</td>
<td>527</td>
<td>669</td>
<td>1036</td>
</tr>
<tr>
<td>3</td>
<td>731</td>
<td>965</td>
<td>1423</td>
</tr>
<tr>
<td>2</td>
<td>890</td>
<td>1350</td>
<td>2063</td>
</tr>
<tr>
<td>1</td>
<td>890</td>
<td>1350</td>
<td>2063</td>
</tr>
</tbody>
</table>
A.4 Dynamic Analysis Verification

Response spectrum analyses are performed for the MRFs to verify the fundamental period and base shear assumptions made in Section A.1.1 and Section A.1.3, respectively. Half of the building mass is equally distributed at each beam-to-column joint of the frames. The resulting periods for both the RBS moment frame and the replaceable link moment frame are higher than the assumed period of 1.16 seconds. The elastic base shear $V_e$ obtained from the response spectrum analysis divided by $R_dR_o$ for both moment frames are equal to or lower than the assumed base shear of 2137 kN. Thus the initial assumptions were valid. Clause 4.1.8.12.5 stipulates that if the base shear obtained from the dynamic analysis is lower than 80% of the base shear calculated from the static method, the design base shear $V_d$ shall be taken as 80% of the static base shear. Thus the design base shear $V_d$ will be taken as 2137 kN. No changes need to be made to the lateral load distribution.

<table>
<thead>
<tr>
<th>Concept</th>
<th>Period (s)</th>
<th>$V_e/R_dR_o$(kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RBS</td>
<td>1.41</td>
<td>2138</td>
</tr>
<tr>
<td>Link</td>
<td>1.53</td>
<td>1978</td>
</tr>
</tbody>
</table>

Because the replaceable link moment frame experienced larger interstorey drifts than the RBS moment frame during the static analyses, $U_2$ factors are only verified for the replaceable link moment frame. The values of $\Delta_f$ are calculated per Clause 4.1.8.12.6 of NBCC 2005, multiplying the interstorey drifts obtained from the response spectrum analysis by $\frac{V_d}{V_e}$, and taking into account torsional effects and notional loads. The $U_2$ factors thus-obtained are all smaller than or very similar to the values assumed in Section A.2.4. Thus no further design iterations are required for both frames. Modal analysis results for the replaceable link frame is presented in Figure A.4.

<table>
<thead>
<tr>
<th>Level</th>
<th>$\Delta_f$(m)</th>
<th>$\Sigma C_f$(kN)</th>
<th>$\Sigma V_f$(kN)</th>
<th>$U_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.0056</td>
<td>7134</td>
<td>651</td>
<td>1.08</td>
</tr>
<tr>
<td>4</td>
<td>0.0082</td>
<td>21143</td>
<td>1401</td>
<td>1.17</td>
</tr>
<tr>
<td>3</td>
<td>0.0098</td>
<td>35153</td>
<td>1987</td>
<td>1.23</td>
</tr>
<tr>
<td>2</td>
<td>0.0105</td>
<td>49163</td>
<td>2409</td>
<td>1.29</td>
</tr>
<tr>
<td>1</td>
<td>0.0099</td>
<td>63226</td>
<td>2667</td>
<td>1.32</td>
</tr>
</tbody>
</table>
Figure A.4 Modal analysis results from SAP 2000
APPENDIX B: REVIEW OF SPECIMEN DESIGN

The test specimens represent typical first floor exterior beam-column connections from the five-storey building designed in Appendix A. The specimens are designed according to CAN/CSA-S16.01, in consultation with Moment Connections for Seismic Applications (CISC, 2005) and Design of Reduced Beam Section (RBS) Moment Frame Connections (Moore et al., 1999). All W-sections are CSA G40.21-350W grade steel ($F_y=350\text{MPa}$, $F_u=450\text{MPa}$). All plates are CSA G40.21-300W grade steel ($F_y=300\text{MPa}$, $F_u=450\text{MPa}$). All bolted connections used 1” A490 bolts.

B.1 MEMBER DESIGN

For each specimen, the link section is chosen according to the moment demand at the plastic hinge location in the five-storey building. All other components of the specimen are capacity designed for the probable moment capacity of the link, based on the geometric configuration of the test set-up. Due to laboratory facility constraints, the distance from the column center to the beam inflection point (point of loading) is shortened from 4.5m to 3.51m. Thus the test specimens experience higher shear forces than expected in the prototype five-storey building where the links are yielding in flexure.

The design of the test specimens is iterative, hence detailed calculations are not presented in this section. Instead, the general design process is described, and the final results are tabulated.

B.1.1 Link Sections

An initial assumption is made for the distance from the column center to the plastic hinge. The link sections are then chosen to resist the moment demand at the plastic hinge location, $M_{f,hinge} = 593 \text{kN-m}$. Table B-1 summarizes the link section properties. Both link types are fabricated from Class-1 W-sections. For the double channel links, the flanges on one side of the W-sections are trimmed flush with the web. The probable moment capacity for each link, $M_{pr}$, is calculated according to Eq. (B.1), adapted from Section 3.1 of Moment Connections for Seismic Applications (CISC 2005):
where \( C_{pr} = \frac{(F_y + F_u)}{2F_y} = 1.14 \)

### TABLE B-1 Link Section Properties

<table>
<thead>
<tr>
<th>Section</th>
<th>Depth ( d )</th>
<th>Flange Width ( b )</th>
<th>Flange Thickness ( t )</th>
<th>Web Thickness ( w )</th>
<th>Section Modulus ( Z_{link} )</th>
<th>Design Moment ( M_r )</th>
<th>Probable Moment ( M_{pr} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>End-plate Link W460x128</td>
<td>467</td>
<td>282</td>
<td>19.6</td>
<td>12.2</td>
<td>3050</td>
<td>947</td>
<td>1339</td>
</tr>
<tr>
<td>Double cut Channel Link W460x89</td>
<td>463</td>
<td>101.5x2</td>
<td>17.7</td>
<td>10.5x2</td>
<td>2560</td>
<td>795</td>
<td>1124</td>
</tr>
</tbody>
</table>

#### B.1.2 Beam and Column Sections

The beam and column sections are selected to resist the probable moment and shear forces induced by plastic hinging of the links. The strength demand at critical sections due to these forces are shown in Figure B.1, and calculated according to Equations (B.2) and (B.3), from Section 3.3 of Moment Connections for Seismic Applications (CISC 2005). Because the replaceable link concept aims to limit inelastic action to the link region, both the beam and the column are intended to remain linear elastic. The beam is designed using Eq. (B.4), to limit flexural yielding at the beam-stub-to-column connection. The column is designed using Equations (B.5) and (B.6), from Clause 27.2.3.2 of CAN/CSA-S16-01 (CSA 2001), which enforces the strong column, weak beam design criteria.

Moment at column face: \( M_{cf} = M_{pr} + V_hx \) \hspace{1cm} (B.2)

Moment at column center: \( M_c = M_{pr} + V_h\left(x + \frac{d}{2}\right) \) \hspace{1cm} (B.3)

Beam capacity design: \( \phi F_y Z_b \geq M_{cf} \) \hspace{1cm} (B.4)

Column capacity design: \( \sum M_{rc} \geq M_c \) \hspace{1cm} (B.5)

where \( M_{rc} = 1.18\phi M_{pc}\left(1 + \frac{C_f}{\phi C_f}\right) \leq \phi M_{pc} \) \hspace{1cm} (B.6)
In all the equations, the subscripts ‘b’ and ‘c’ denote beam and column, respectively. Because axial force is not imposed on the column during the experiments, $C_f$ is taken to be zero in Eq. (B.6).

Both the beam and the column are over-sized to ensure that they will remain elastic through all four tests. This over-design also represents more realistic section requirements for storey drift control. Table B-2 shows the final beam and column section properties. These section properties will be used to determine the location of the links. When the specimen geometry is finalized and the strength demands at the critical sections can be accurately assessed, the beam and column capacities will be verified.

**TABLE B-2** Beam and Column Section Properties

<table>
<thead>
<tr>
<th>Section</th>
<th>Depth $d$ (mm)</th>
<th>Flange Width $b$ (mm)</th>
<th>Flange Thickness $t$ (mm)</th>
<th>Web Thickness $w$ (mm)</th>
<th>Section Modulus $Z_x$ ($10^3$ mm$^3$)</th>
<th>Plastic Moment $M_p$ (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>695</td>
<td>355</td>
<td>24.8</td>
<td>15.4</td>
<td>7610</td>
<td>2664</td>
</tr>
<tr>
<td>Column</td>
<td>416</td>
<td>406</td>
<td>48.0</td>
<td>29.8</td>
<td>7970</td>
<td>2790</td>
</tr>
</tbody>
</table>

Figure B.1 Strength demands at critical sections: (a) column face, (b) column center
B.1.3 Link Location and Length

Before determining the distance from the column face to the link connection, \( a \), and the length of the link, \( s \), (both illustrated in Figure B.2), recommended limits for Reduced Beam Sections (RBS) are considered. The upper limits keep the two parameters small to minimize moment increase between the plastic hinge and the face of the column. The lower limits ensure that \( a \) is large enough to permit stress in the RBS to spread uniformly across the beam flange width at the column face, and that \( s \) is large enough to avoid excessive inelastic strains within the RBS (Moore et al. 1999). The recommended limits are calculated according to Equations (B.7) and (B.8), from Section 6.1 a) of Moment Connections for Seismic Applications (CISC 2005). Since the plastic hinges form in the replaceable links, limits of \( s \) are calculated based on the depth of the links instead of the depth of the beam.

\[
0.5b_b \leq a \leq 0.75b_b \tag{B.7}
\]

\[
0.65d_{link} \leq s \leq 0.85d_{link} \tag{B.8}
\]

Table B-3 summarizes the recommended RBS values and the chosen values for \( a \) and \( s \). Figure B.2 illustrates the final length and location of the test specimens.

<table>
<thead>
<tr>
<th>TABLE B-3</th>
<th>Link Length and Location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( a )</td>
</tr>
<tr>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>Recommended</td>
<td></td>
</tr>
<tr>
<td>Limits</td>
<td>178 ~ 266</td>
</tr>
<tr>
<td>End-plate link</td>
<td>200</td>
</tr>
<tr>
<td>Double C Link</td>
<td>220</td>
</tr>
</tbody>
</table>
B.1.4 Panel Zone Design

The horizontal shear resistance of the panel zone is checked against the shear demand on the panel zone, according to Clause 27.2.4.3(a) of CAN/CSA-S16-01 (CSA 2001). The shear resistance is predicted using Eq. (B.9) to eliminate yielding in the panel zone. The shear demand is calculated according to Eq. (B.10). A 12mm single doubler plate is added to each panel zone, resulting in a total panel zone thickness $w'$ of 41.8mm.
Continuity plates are deemed necessary next to the panel zone region because the column flange thickness, \( t_c = 48 \text{mm} \), is less than the greater of Equations (B.11) and (B.12), both taken from Section 3.5 of Moment Connections for Seismic Applications (CISC 2005). 16mm thick continuity plates are used to transfer forces in the beam-stub flanges into the column. The doubler plates are trimmed flush with the continuity plates. Both the continuity plates and the doubler plates are prepared and welded according to Section 7 of Moment Connections for Seismic Applications (CISC 2005).

\[
V_r = 0.55 \phi d \psi w' F_{yc} \quad \text{(B.9)}
\]

\[
V_{pz} = \frac{M_c}{d \psi z t_b} \quad \text{(B.10)}
\]

\[
0.4 \sqrt{1.8 b_b t_b \frac{R_{yz} F_{yb}}{R_{yc} F_{yc}}} = 50 \text{mm} \quad \text{(B.11)}
\]

\[
\frac{b_b}{6} = 59 \text{mm} \quad \text{(B.12)}
\]

The strength demands at critical sections are recalculated and checked against the beam and column capacities. The results are summarized in Table B-4.

<table>
<thead>
<tr>
<th>TABLE B-4</th>
<th>Strength Demands and Capacities at Critical Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( M_{pz} )  ( V_h )  ( M_{cf} )  ( \Phi F_{yz} Z_b )  ( M_c )  ( \sum M_{xc} )  ( V_{pz} )  ( V_r )</td>
</tr>
<tr>
<td>Endplate Link</td>
<td>1339</td>
</tr>
<tr>
<td>Double C Link</td>
<td>1124</td>
</tr>
</tbody>
</table>

\( \Phi \) and \( F_{yc} \) values are taken from CISC (2005).
B.2 CONNECTION DESIGN

B.2.1 End Plate Link Connection

An eight-bolt, extended-end plate design is chosen for the connection, as shown in Figure B.3. The connection is designed according to Section 4 of Moment Connections for Seismic Applications (CISC 2005) as well as research by Sumner (2003).

The end plate length and width are determined according to the outer dimensions of the beam. The end plate thickness is calculated according to Eq. (B.13), from Section 4.1 of Moment Connections for Seismic Applications (CISC 2005), to preclude flexural yielding of the plate under the probable moment experienced by the link. The equation is based on yield line analysis performed by Sumner (2003). End plates with thickness of 41.3mm are used for the link. To ensure that the link would yield first, 50mm thick end plates are used for the beam and beam-stub.

![End plate connection geometry](image_url)
where \( s = \sqrt{B_p g} \)

and \( F_{yp} = 250 \text{MPa} \)

The capacities of individual bolts are calculated according to Equations (B.14) and (B.15), from Clauses 13.12.1.1 and 13.12.1.2 of CAN/CSA-S16-01 (CSA 2001), respectively. The connection capacity is verified according to Eq. (B.16) to preclude bolt tension rupture, taken from Section 4.1 of Moment Connections for Seismic Applications (CISC 2005). Because the per-bolt shear demand is very small, there is minimal moment-shear interaction.

Single Shear Capacity: \( V_r = 0.6 \phi_{vm} A_v F_u = 252 kN \) (B.14)

Tension Capacity: \( T_r = 0.75 \phi_{vm} A_v F_u = 314 kN \) (B.15)

\[
\frac{T_r}{\phi_{vm}} = 393 kN \geq \frac{M_{pr}}{4(h_3 + h_1)} = 374 kN
\] (B.16)

Bolt shear failure and end plate shear yielding do not govern, thus their calculations according to Section 4.1 of Moment Connections for Seismic Applications (CISC 2005) are not shown.

To help transfer link flange forces into the beam, stiffeners equal to link flange width and thickness are provided on the beam and beam-stub.

**B.2.2 Double Channel Link Connection**

The double channel links are connected to the beam and beam-stub using eccentric bolted connections, shown in Figure B.4.
The connection is designed for the probable moment experienced by the link, $P_r = 364$ kN load ($P_r$) applied with an eccentricity of 3.09m (the distance from the point of loading to the center of the connection). The method of instantaneous center of rotation is used with the load-deformation relationship shown in Eq. (B.17). The coefficient $C$, which relates the connection demand to demand in each bolt, is found to be 0.84. Using Eq. (B.18), the per-bolt demand, $R_f$, is calculated to be 434kN.

$$R = R_d(1 \angle e^{\frac{\mu \Delta}{\lambda}})^\lambda$$  \hspace{1cm} (B.17)

where $\mu = 10.0$, $\lambda = 0.55$, $\Delta_{\text{max}} = 0.34\"$

$$P_f = CR_f$$  \hspace{1cm} (B.18)

The shear capacity of the bolts is adequate according to Clause 13.12.1.1 of CAN/CSA-S16-01 (CSA 2001), shown in Eq. (B.19). In order to minimize bolt hole deformation, a modified equation based on Clause 13.10 (c) of CAN/CSA-S16-01 (CSA 2001) is used to calculate bearing capacity. Using Eq. (B.20), doubler plates are deemed necessary for both the link webs and the beam web. Doubler plates (6mm) are added to the connection regions of the link, and two plates (10mm and 12mm) are added to either side of the beam and beam stub connections. The resulting bearing
capacity is 516kN/bolt for the links and 562kN/bolt for the beam and beam stub. The doubler plates are fillet welded all around, with weld sizes equal to the doubler plate thickness.

\[
V_r = 0.6 \phi_v m A_y F_y = 504 \text{kN} \geq 434 \text{kN} \tag{B.19}
\]

\[
B_r = 2 \phi_b t d_{db} F_y = 516 \text{kN} \geq 434 \text{kN} \tag{B.20}
\]
APPENDIX C: FABRICATION DRAWINGS

This section presents the original specimen design drawings prepared by N. Mansour, as well as modification drawings prepared by the author.
<table>
<thead>
<tr>
<th>PC MARK</th>
<th>Q'TY</th>
<th>DESCRIPTION</th>
<th>WEIGHT (kg)</th>
<th>REM ARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF1</td>
<td>4</td>
<td>Replaceable Link Type 1</td>
<td>359</td>
<td></td>
</tr>
<tr>
<td>MRF2</td>
<td>2</td>
<td>Replaceable Link Type 2</td>
<td>392</td>
<td></td>
</tr>
<tr>
<td>MRF11</td>
<td>1</td>
<td>Connecting Column</td>
<td>Δ 1,822</td>
<td></td>
</tr>
<tr>
<td>MRF12</td>
<td>1</td>
<td>Connecting Beam</td>
<td>1,008</td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td></td>
<td><strong>3,581</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Marking and Description Table

<table>
<thead>
<tr>
<th>Mark</th>
<th>Q'TY</th>
<th>Description</th>
<th>Length</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>pb</td>
<td>12</td>
<td>W360x382</td>
<td>3900</td>
<td></td>
</tr>
<tr>
<td>ma</td>
<td>1</td>
<td>L = 150</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>pc</td>
<td>1</td>
<td>L = 690x217</td>
<td>695</td>
<td></td>
</tr>
<tr>
<td>pd</td>
<td>4</td>
<td>L = 180x10 x 140</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>mb</td>
<td>1</td>
<td>L = 400</td>
<td>400</td>
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</tr>
<tr>
<td>pe</td>
<td>1</td>
<td>L = 550</td>
<td>550</td>
<td></td>
</tr>
</tbody>
</table>

**Remarks:**
- All holes are 1 1/16" UON
- Weld Electrodes are E480XX

**University of Toronto**

**MRF Column with Link Connection Stubs**

**REV 2 22/09/2005 | Flange weld detail revised as shown.**

---

**APPENDIX C**

**SEISMIC PERFORMANCE OF STEEL MOMENT-RESISTING FRAMES WITH NONLINEAR REPLACEABLE LINKS**

**188**

**For Notch Detail See SK-1 (Typ 6 Places)**

**6-PL-pb**

(NS & FS)

**W360x382 x 3900**

(Top & Bottom)

**SFI**

**2/90/22**

**04/AUG/2005**
**APPENDIX C**

**SEISMIC PERFORMANCE OF STEEL MOMENT-RESISTING FRAMES WITH NONLINEAR REPLACEABLE LINKS**

---

**SECTION A-A**

For both panels, first weld plate to column flanges on both sides, fill column fillet until flush with the plate (650mm weld). Then weld plate to top and bottom stiffeners (320 mm weld).

---

**W360 x 382 COLUMN**

QTY = 1

---

**W690 x 217 BEAM**

QTY = 1

- Weld Electrodes are E49xx

---

**CHANNEL LINK**

QTY = 2

---

**University of Toronto**

**TITLE:**

Additional Welds For MRF

**DWG. NO.**

MRF 14

**REV**

2
## Summary of Additional Plates to be Welded

<table>
<thead>
<tr>
<th>Plate Label</th>
<th>Description</th>
<th>Quantity</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL-pa</td>
<td>Panel zone doubler plates. One in each panel region, on one side of the column web only.</td>
<td>2</td>
<td>For both panels, first weld plate to column flanges on both sides, fill column fillet until flush with the plate (650mm weld). Then weld plate to top and bottom stiffeners (320mm weld).</td>
</tr>
<tr>
<td>PL-pb</td>
<td>Column end plates. On both ends.</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>PL-pc</td>
<td>Doubler plates for beam sections. One for the beam and one for the beam stub on the column, on one side of the web only.</td>
<td>2</td>
<td>10mm doubler plates have already been welded to the other side of the beam section</td>
</tr>
<tr>
<td>PL-pd</td>
<td>Doubler plates for channel links. two on each link.</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

* All plates have been prepared and will be tack-welded in place before final welding